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Wood Engineering and Seismic Analysis in Building Structures

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SUMMARY

Title: Wood engineering and Seismic Analysis on Building Structures

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Key Words: Earthquake, building, seismic design, wood, and Glulams

Earthquakes have caused many disasters along with multiple casualties throughout history. Nevertheless, as science has evolved and a better understanding of the nature of earthquakes has been achieved, new methods of designing building have developed. Human safety has always been and is the biggest concern, and that is what makes seismic design so important when designing any building.

Earthquake Engineering is going to be the first topic addressed in this thesis, spanning from the most basic seismic concepts to how a seismic equivalent force is applied to any building. Plate tectonics theory is briefly explained as well as the significance of the Richter scale. These are general seismic concepts that can be omitted for structural design purposes but convenient to know and understand.

The biggest focus on the Earthquake Engineering chapter is explaining the fundamentals of structural dynamics and how this leads to the seismic equations provided by current codes. The different degrees of structural dynamics complexity are exposed and commented on (linear vs non-linear) and a thorough explanation of all the different factors taken into consideration by current codes are explained in this section.

The expected substantial growth of the world's population calls for a wise use of our resources when it comes down to building structures. Wood is a renewable material and the best option from an environmental point of view, which is why special attention to its structural and environmental properties should be paid.

This thought brings us to the next core section of this thesis, which is wood engineering. This section starts explaining the very basic properties of wood and how they are taken advantage of in wood engineering. The two most basic types of wood products, which are sawn lumber and Glulams, are analyzed. Their fabrication processes, as well as their grades, species types and adjustment factors for structural design are going to be investigated.

The last core section is the Case Study. All of the concepts and ideas exposed up to this point are combined together in a one story framed wood building. The main gravity wood members are designed using the principles exposed in the wood engineering section. The last part of the Case Study focuses on the first core chapter, which is Earthquake Engineering; a seismic analysis based on the principles and methods explained in this chapter is performed. All of the results are well documented and analyzed.

The thesis ends providing a list of conclusions commenting on the most important aspects that have been learned. Special attention is given to the importance of having a

good understanding of how seismic analysis works and to learn how we can use wood in applications where we would typically use concrete or steel as our structural resistant material.

Title: Wood engineering and Seismic Analysis on Building Structures

Author: Francisco Javier Gonzalez Ramirez

Tutor: Horia Alejandro Barbat

Key Words: Terremoto, edificio, diseño sísmico, madera y Glulams

Durante la historia, los terremotos siempre han sido sinónimo de destrucción y de muerte. A medida que nuestro conocimiento y comprensión de los fenómenos que causan los terremotos ha aumentado junto con la evolución de la ciencia, se han creado nuevos métodos para diseñar edificios contra aquéllos. Proteger vidas es siempre la prioridad cuando diseñamos cualquier edificio y de ahí la importancia del cálculo sísmico.

La ingeniería sísmica constituirá la primera materia de esta tesis, en la que se analizaran una amplia serie de conceptos, yendo de los conceptos más básicos de la ingeniería sísmica hasta la explicación de cómo se calcula la fuerza lateral equivalente que se aplica a un edificio. Se expondrá una breve explicación de la teoría de la tectónica de placas y del significado de la Escala Richter, tratándose de conceptos de carácter muy general que, aunque podrían ser omitidos en relación al cálculo sísmico, su comprensión resulta de utilidad.

El capitulo de Ingeniería sísmica se centrará en la explicación de las bases de la dinámica de estructuras y en como se deducen las ecuaciones que dan los códigos actuales. Se explicarán y compararán los distintos grados de complejidad dentro de la dinámica de estructuras (análisis lineal vs no lineal). Finalmente se analizarán todos los factores usados por los códigos actuales para calcular las fuerzas sísmicas equivalentes.

Se espera que la población mundial siga creciendo y por ello debemos utilizar los recursos de los que disponemos de la mejor forma posible. La madera es un material renovable y la mejor alternativa desde un punto de vista ambiental, por estos motivos debemos prestar atención a las propiedades estructurales y ambientales que la madera tiene que ofrecer.

Esta reflexión nos lleva directamente al siguiente capítulo clave de esta tesis que es la Ingeniería de la Madera. El capítulo comienza explicando las propiedades más básicas de la madera y de que forma se les puede sacar el mayor rendimiento. Las dos tipologías de productos más básicos, que son la madera serrada y los Glulams, son objeto de análisis. Se van a examinar sus procesos de fabricación, junto con sus grados, tipo de especies y factores de ajustamiento para el diseño estructural

El último gran capítulo es el ejemplo práctico. Todos los conceptos previamente

expuestos van a ser aplicados a un edificio de madera de un piso. Los elementos encargados de resistir las fuerzas de gravedad van a ser diseñados usando las ecuaciones y principios explicados en el capitulo de Ingeniería de la Madera. La parte final del ejemplo práctico se centra en el otro gran capítulo de esta tesis, que es la Ingeniería Sísmica: Se va a hacer un análisis sísmico basado en los principios y métodos expuestos. Todos los resultados van a ser explicados y comentados.

La parte final de esta tesis son las conclusiones, donde se resumen los aspectos más importantes que se pueden extrapolar después de la realización de esta tesis, valorando la importancia que tiene la comprensión del análisis sísmico y el potencial que tiene la madera en ciertas aplicaciones donde típicamente se usaría hormigón o acero como material estructural.

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1 INTRODUCTION

1.1. Introduction and motivation

The tree and its wood have played a prominent role in human life throughout history. Wood has been one of our most important building materials from early Paleolithic times, both for building and for the manufacturing of tools, weapons, and furniture. From the earliest times, the use of wood involved consideration of quality, cost and availability, as well as the intended use of the product. Scarcity of valuable timber led to careful and economic use. Boards were carefully matched and fitted; blemishes were removed and filled. Practices that begun many centuries ago are still carried over, with refinement, to the current use of wood for high quality applications. Early humans used wood because it was available and no elaborate tools were needed to work it. In the early days, however, the quality of the products depended more on the quality of the wood and the skill of the workman than on the tools available for woodworking. The development of copper tools by about 5000 BC opened new opportunities for craftsmanship – opportunities that have been carried forward to this day.

Wood has been the most versatile and useful construction material for thousands of years and is still used more than any other construction material. The style and durability of structures built at various times and places have depended on the type and quality of timber available and the conditions of use, as well as the culture and way of life of the people concerned. In forested zones, where timber was plentiful, solid walls were built of tree trunks or heavy timbers. Splitting logs and setting them vertically in the ground or on a sill plate on the ground frequently made timber houses in Neolithic Europe. Also thousands of years old is the concept of construction with logs placed horizontally, as in a log cabin. It has been used most frequently in the northern, central, and mountainous area of Europe and North America where there have been plentiful supplies of large, straight trees. As construction with stone and concrete became common, wood was used for concrete forms and supplementary structural components such as trusses and roof supports. Wood construction has had an interesting evolution in North America because of the relatively abundant timber resource and the scattered development of much of the country. Native Americans built homes of poles or planks.

Nowadays, the widespread use of wood in the construction of buildings has both an economic an aesthetic basis. The ability to construct wood buildings with a minimal amount of specialized equipment has kept the cost of wood frame buildings competitive with other types of construction. On the other hand, where architectural considerations are important, the beauty and warmth of exposed wood are difficult to match with other materials.

From an environmental point of view, wood construction is the preferred material to build by architects. Steel construction alone represents a 3% of the energy and greenhouse effects on the planet, whereas concrete represents a 5%. Wood actually stores carbon dioxide, so it contributes negatively to the emission of greenhouse emissions, making it a great material when thinking green. The environmental perspective is going to be taken into consideration with the significant future growth of the world's population in the coming years, since all of these people are going to require buildings for living, working and recreational purposes.

Wood has historically been used for one and two story buildings such as single-family residences and retail, but never to design skyscrapers. Steel and concrete have been the structural materials of choice for these kinds of buildings, and one of the reasons for such thing has been the limited workability and size that wood has had in the past. Nowadays, we have this new technology called wood engineered products, where small pieces of wood are assembled in a way they create considerably strong members with structural capacities comparable to that of a concrete or steel member. This has revolutionized modern construction, and new directions to building high with wood have already been taken with many high-rise wood buildings in the horizon.

I would like to mention architect Michael Green in my introduction. This Canadian architect is very well known because of his investigations with wood and his belief of the feasibility of wood construction up to 30 stories. His company (Michael Green Architects) has designed the tallest wood building in North America, consisting of 9 stories. It is important to remark in this achievement that most building codes limit wood construction to 5 stories. The building is called "Wood Innovation Centre" and it is located in Prince George, British Columbia (Canada). The building is currently under construction and it is expected to be open in September 2014.

When used as a material for structural design, wood differs from conventional steel, concrete or light gage design in many ways. Wood is an anisotropic material, that is, its properties change with the orientation of its axis, and that plays a very important role when designing different parts of the structure for forces acting in different directions. Understanding how wood strength is calculated is not an easy task, and providing the fundamentals to understanding the methods used to do so as well as all the different factors that affect its strength is going to be one of the goals of this thesis.

There are several reasons that drove me to pick the topic of wood and seismic analysis for this thesis;

The shortage of wood as a structural material for construction in Spain made me really enjoy learning about a different material from the ones we are used to hearing about, which are structural steel and concrete. Its widespread use in America impressed me and I thought that analyzing wood from the most basic level, exposing its structural characteristics and finally using them in a real world structural problem would be very interesting and challenging.

Being able to design a building with wood that is just as safe as any concrete and steel building stunned me, especially since we tend to think that wood would typically be used at a smaller scale and never used as the only structural material to construct a building. These are the main reasons that motivated and drove me to keep expanding my knowledge on wood and to include this topic in my thesis.

Designing structures against earthquakes was fascinating to me when I first learned about seismic design due to many reasons; I have always heard and followed the major earthquakes that have happened and that are happening on earth, but living in a low hazard seismic area makes us lose awareness of how important designing against earthquake forces is, and that is something that I learned in the beginning of my career in North America.

Being able to design bridges and buildings capable of withstanding earthquakes up to 8.0 in the Richter scale is extremely impressive to me, especially when it comes to skyscrapers, and I wanted to include this in my thesis in a smaller scale showing how a simple one-story building is designed.

Finally, I figured that I could combine both topics in a case study, showing how a structure can be designed using wood as the main structural material and subjecting the structure in question to a design earthquake, putting all of the explained concepts of my thesis into practice and helping the reader understand all of the theory behind it.

1.2. **Objectives**

There are two main personal objectives that I intend to achieve before starting this thesis:

My first main objective is to show that wood is a structural material that can be used in a wide range of applications and that it can be more economical than steel and concrete for buildings of small dimensions. In order to achieve this goal I am going to show how to calculate the structural properties needed to perform structural strength calculations based off a piece of wood from a specific wood species.

Showing how modern codes come up with their seismic equations and the significance of all of the different factors that affect its value consists in my second main goal. A good understanding of its nature and of the different modification factors is crucial when applying seismic equations, which is why this is going to be the other main focus of my thesis.

Finally, my last objective is to show all of the steps that have to be followed and all the different factors that have to be calculated to perform a seismic analysis to a wood structure by using them in a real life example.

2 EARTHQUAKE ENGINEERING

2.1. **Basic seismology**

2.1.1. OVERVIEW

An earthquake is an oscillatory, sometimes violent movement of the Earth's surface that follows a release of energy in the Earth's crust.
A sudden dislocation of segments of the crust, volcano eruption, or a man-made explosion can generate this energy. Dislocations of the crust, however, cause most of the destructive earthquakes.

When subjected to geologic forces from plate tectonics, the crust initially strains elastically (bends and shears). As rock is stressed, it stores energy.

When the stress exceeds the ultimate strength of the rock, the rock breaks and quickly moves into new positions. In the process of breaking, the strain energy is released and seismic waves are generated. This is the basic description of the elastic rebound theory of earthquake generation.

These waves travel from the source of the earthquake (known as the hypocenter or focus) to more distant locations along the surface of and through the Earth. It is important to take into consideration that the speeds at which these waves travel depend on the nature of the waves and the material that they travel through.

2.1.2. GLOBAL SEISMICITY

Most earthquakes occur in areas bordering the Pacific Ocean. This circum pacific belt, nicknamed the ring of fire, includes the Pacific coasts of North America and South America, The Aleutian Islands, Japan, Southeast Asia and Australia. The reason for such concentration can be explained by plate tectonics, which will be explained in later sections. See Figure 1 on the next page.

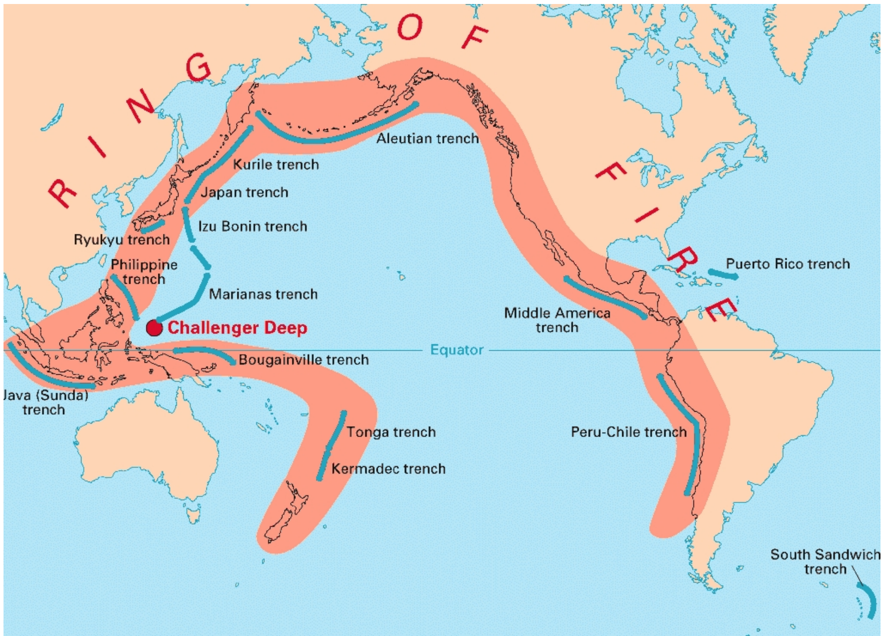

Figure 1. The ring of fire.*Source: United States Geological Survey(USGS).*

The United States has experienced less destruction than other countries located in this earthquake zone and this is partly due to the countries young age and attention to earthquake resistant construction methods.

2.1.3. CONTINENTAL DRIFT

It has been known since the early 1900s that the continents are moving relative to one another, movement known as continental drift. In fact, fossilized records of past climates indicate that the continents have been moving slowly about the globe for millions of years. For example, the same 300 million year old fossilized deposits are found in India and in the Artic.

The theory of continental drift was reasonably established during the 1930s, but was not universally accepted. In the 1950s, the emerging science of paleomagnetism provided new supporting evidence of continental drift. Many rocks, such as volcanic rock solidified from molten lava, contains tiny grains of magnetic minerals such as magnetite. When these minerals are formed, they retain the magnetic orientation of the earth at the time of their formation. The magnetic orientations of rocks suggest the same ancient locations of the continents suggested by paleoclimatology and geologic criteria.

2.1.4. PLATE TECTONICS

Most earthquakes are a manifestation of the fragmentation of the Earth's outer shell (known as the lithosphere) into various small and large plates. There are seven very large plates, each consisting of both oceanic and continental portions.
Each plate is approximately 80 to 100km thick and has thin and thick parts. The thinner part deforms by elastic bending and brittle breakage. The thicker part yields plastically.

Beneath each layer there is a viscous layer on which the entire plate slides. The plate themselves tend to be internally rigid, interacting only at the edges.

Shallow earthquakes represent sudden slippages and are accompanied by a release of elastic energy stored in the rock over a long period. It is not totally clear whether the deep mantle subduction zone earthquakes are accompanied by similar elastic releases or are merely abrupt contractions of the part of the subducting plate into rock of higher density. See Figure 2 for an illustration of the subducting process.

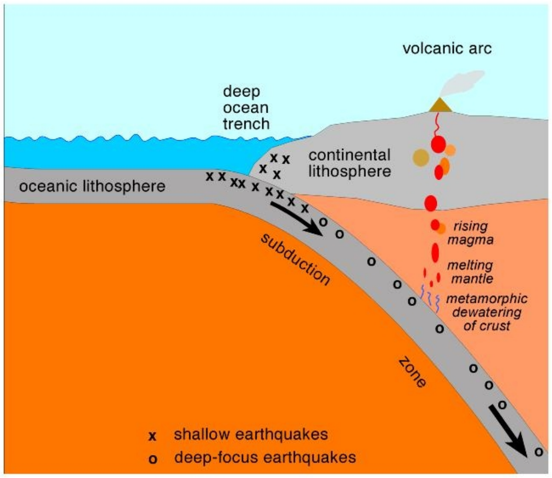

Figure 2. Subduction process.*Source: United States Geological Survey(USGS)*

Only a fraction of the energy released in an earthquake actually appears in seismic waves. Most of the released strain energy is reabsorbed locally by the moving, deforming, and heating of the rock. The fraction absorbed increases irregularly with increasing size of earthquake. Minor earthquakes generally do not represent a sufficient release of energy to dissipate the strain energy and prevent great earthquakes, although a slow creep along a fault can provide a partial release.

Great earthquakes occur primarily along convergent plate boundaries. Submerged ridges are so hot at relatively shallow depths that the solid rock above them cannot store enough elastic strain energy to produce great earthquakes.

2.1.5. RICHTER MAGNITUDE SCALE

In 1935, Charles F. Richter in the California Institute of Technology developed the Richter magnitude scale to measure earthquake strength. The magnitude, M, of an earthquake is determined from the logarithm of the amplitude recorded by a seismometer. Adjustments are included in the magnitude to compensate for the variation in the distance between the various seismometers and the epicenter. Because the Richter number is a logarithmic scale, each whole increase in magnitude represents a ten fold increase in the measured amplitude.

The Richter magnitude is expressed in whole numbers and decimal fractions. For example, a magnitude of 5.3 might correspond to a moderate earthquake. A strong earthquake might be rated at 7.5. Earthquakes with magnitudes of 2.0 or less are known as micro earthquakes. While recorded by seismometers, micro earthquakes are rarely felt by people at all.

Several thousand seismic events with magnitudes of 4.5 or greater occur each year and are strong enough to be recorded by seismometers all over the world. Earthquakes of this size and below have little potential to cause damage. Great earthquakes, such as the 1906 San Francisco earthquake occur once a year on average.

The magnitude of an earthquake depends on the length and breadth of the fault slip, as well as on the amount of slip. The largest examples of fault slip recorded in California accompanied the earthquakes of 1857, 1872 and 1906, all of which had estimated magnitudes over 8.0 on the Richter scale.

See Figure 3 for an illustration of the Richter scale and the levels of damage associated with each value.

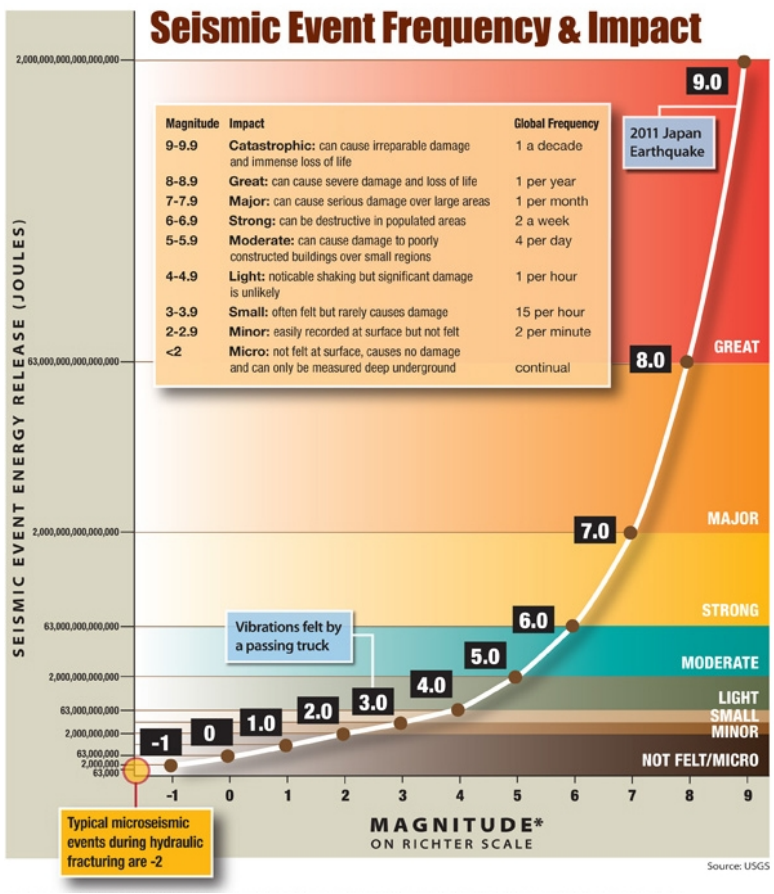

Figure 3.Richter scale levels of damage. *Source: United States Geological Survey(USGS).*

2.2. **Elements of structural dynamics**

The following section's intent is to provide a clear understanding of what seismic forces are and the theory behind it. An explanation of how all structural codes (using the International Building Code 2009 as an example) have developed all their methods to go from a specific ground motion and a structure to a series of loads applied to this last one will be provided at the end of this section.

2.2.1. SEISMIC RESPONSE OF A SINGLE DEGREE OF FREEDOM STRUCTURE

As shown in Figure 4, a single degree of freedom system is used to represent a structure whose motion can be described by a single unknown quantity, which in this case would be the structure's deformation (u).
How this deformation is computed is not an easy task, first we would have to take a look at structural properties such as the stiffness associated with the total deformation (k), mass (m), yielding strength (f_y) and level of damping(C). It is important to note that all of these properties are independent of time.

At any given time during the shaking, say at time t, the ground moves a value of $u_g(t)$, and at the same time the structure has a deformation value of $u(t)$. It is important to remark that all response related quantities (those related to $u(t)$and its derivatives) are a function of time (time dependent).

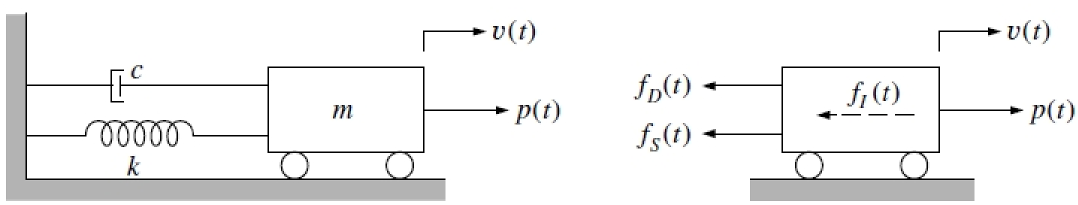

Figure 4. Single degree of freedom. *Source: RayW. Clough&Joseph Penzien. 2003.*

2.2.2. LINEAR ELASTIC RESPONSE AND SPECTRUM

If the ground motion intensity is fairly low or the structure is very strong (high yielding stress value), the structure would not reach its maximum capacity level, and therefore it would remain elastic. The response associated with such systems is called elastic-linear response. The response spectrum describes the relationship of the maximum response of a single degree of freedom structure given a ground motion.

Using Figure 4, we can establish a general equation of motion at time t for our single degree of freedom structure:

$$f_I + f_D + f_S = p(t) \qquad (2.1)$$

Where,

f_I: Inertia force

f_D: Viscous damping force

f_S: Spring force

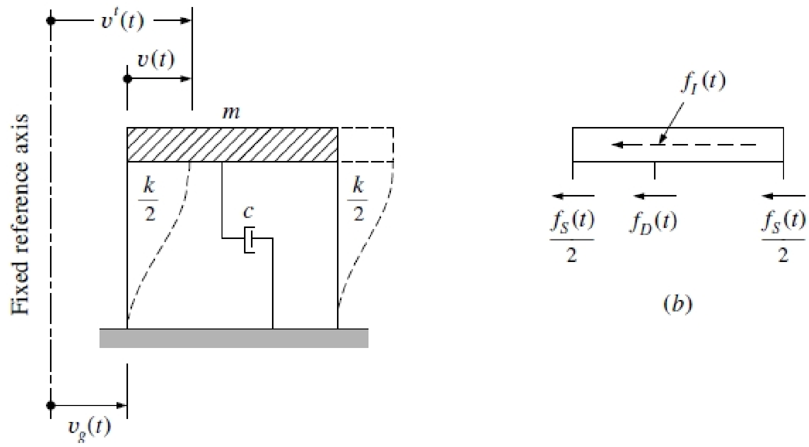

Figure 5.Equation of motion representation.*Source: RayW. Clough & Joseph Penzien. 2003.*

The solution to Equation 2.1 depends on the time dependent function of $u_g(t)$ when m, k and C are given. $u_g(t)$ is often characterized by its peak value, u_{g0} and its frequency contents.
Frequency contents refer to the range of frequencies in $u_g(t)$ time history function, and have profound impact on seismic response of structures.
In earthquake engineering practice, the frequency contents are considered as important as the peak ground accelerations due to its potential to excite the structure by resonant response. The peak ground acceleration on hard soils might be directly related to the magnitude and distance from the epicenter, and usually will be provided to the engineer by codes. Consequently, the most important task for an engineer in deciding the design seismic load is to take into account the effect of the soil deposit at the structural site on the ground motion, and structural response.

Response spectrum or its modified version, design spectrum, is often used as a tool to do such thing. A response spectrum is developed based on the solution to Equation 2.1. Since the function of any $u_g(t)$ is rather chaotic, there is no closed solution for $u(t)$. However, numerical solution to Equation 2.1 is just as effective.

The general procedure to obtain a design spectrum is as follows:

1) Select a ground acceleration time history $u_g(t)$.

2) Define damping ratio, C (usually assumed equal to 5% in design).

3) Select values for m and k to define a new $T_n = 2\pi\sqrt{\frac{m}{k}}$ = natural period of vibration.

4) The total duration of vibration of interest (usually as long as the duration of the ground shaking) is divided into small intervals, often in the order of 0.5% of second (for this elastic dynamic analysis process, and much smaller intervals are needed for inelastic dynamic analysis). A step by step numerical solution to Equation 2.1 will find $u(t)$ at each time interval through the entire duration, from which the maximum value of $u(t)$, D, is identified.

D=Maximum value of $u(t)$

5) Repeat steps (3) and (4) usually for T_n between 0.05 sec and 3 sec.

6) Plot D versus T_n, which is the deformation response spectrum

7) Define S_a, pseudo-acceleration spectrum, as

$$S_a = w_n^2 D \tag{2.2}$$

S_a can be used directly to define $f_{s,max}$, which is the maximum value of the elastic resistant force f_s, when the deformation $u(t)$ reaches its maximum value D, as:

$$f_{s,max} = ku_{max} = m(w_n^2 D) = mS_a \tag{2.3}$$

$f_{s,max}$ will be treated as an equivalent static lateral force applied to the structure to obtain maximum response items such as bending moment and axial force in structural members. Note that S_a is just a symbol for $w_n^2 D$, and has no relationship with acceleration response. It is called pseudo-acceleration spectrum only because it has units of acceleration. As shown by the solid curve in Figure 6, there are many sharp peaks and valleys, particularly for T_n < T_s, which indicates that a small change in structural properties (in terms of period of vibration), particularly for low-rise buildings and short span bridges, would result in a significant difference in S_a. This is due to the fact that any given ground motion has a wide range of frequencies, and any change in structural properties such as mass, stiffness, or both would result in significant alteration in response. When the natural period of vibration of a structure can be determined pretty accurately (say, within the tenth of a second), the response from the spectrum should be the same as the time analysis result one (directly from the solution of Equation 2.1). However, for the purpose of new design, when the value of the period can only be roughly estimated before the structure is even built, a smooth curve, as indicated by the dash line of Figure 6, is used.

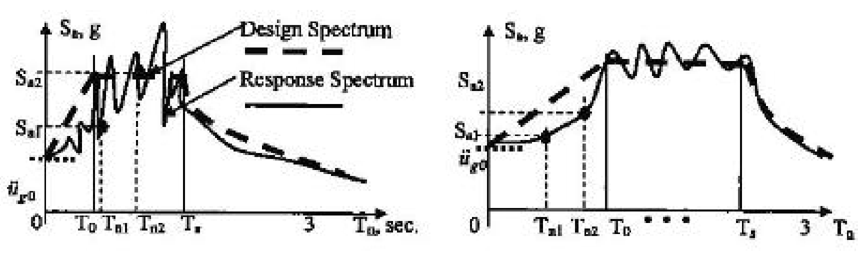

Figure 6.Design and response spectrum. *Source: Chopra, Anil K. 2011.*

Figure 6 graphically illustrates the spectrum process. Figure 7 shows the ground motion on hard soil/rock and the response of different structures with different periods of vibration. Figure 7 also shows the ground motion on soft or deep soil deposit and the response of different structures. Figure 8 shows the deformation spectrums for the two selected ground motions, respectively, by connecting several maximum responses, including D_1 and D_2. Pseudo acceleration spectra is constructed by multiplying plots in Figure 8 by ω_n^2 .

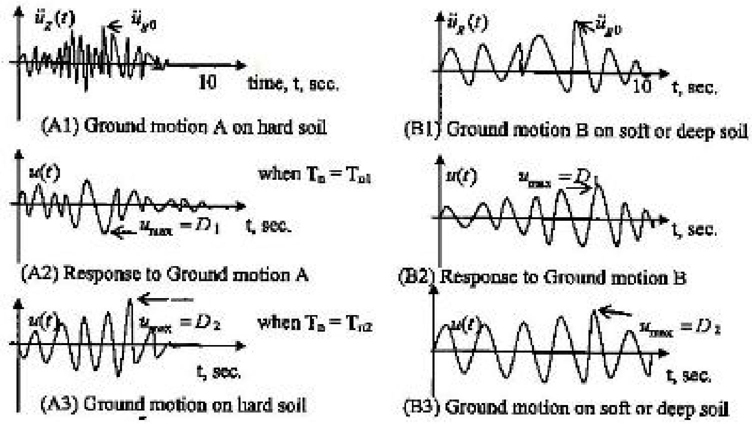

Figure 7. Ground motion on hard and soft soil. *Source: Chopra, Anil K. 2011.*

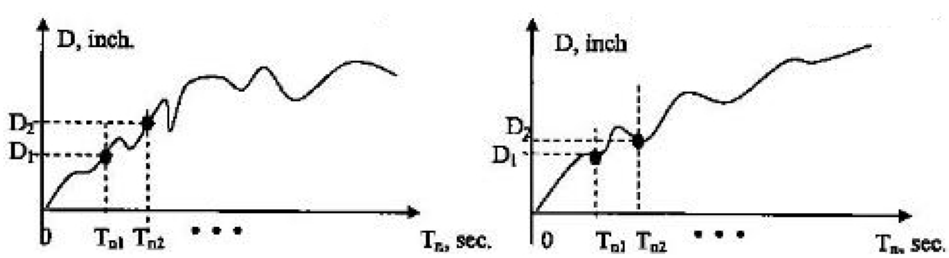

Figure 8. Deformation spectrums. *Source: Chopra, Anil K. 2011.*

The acceleration response spectra (drop pseudo), as shown in Figure 6, is convenient for design practice, and its characteristics can be highlighted as follows.

1) There are three distinctive sessions in the spectrum with (i) $T_n < T_0$, (ii) $T_0 < T_n < T_s$, and (iii) $T_n > T_s$.

2) **<u>Rigid Structure Session:</u>** The structure with $T_n < T_0$ is very rigid (with a large stiffness or a little mass). Some typical structures in this category include low residential buildings, extremely short span highways bridges and alike. The extreme case is a rigid body (K=∞ and $T_n = 0$) connected to the ground, which results in $S_a = u_{g0}$. In design practice, a structure having) $T_n < 0.05\ s$ is considered a rigid body. The

mean values of S_a(neglecting the peaks and valleys) tend to increase as T_n changes from 0 to an approximate value of T_0. The value of T_0 mainly depends on the frequency contents in the ground motion. T_0 is usually small, in the order of 0.1 to 0.2 second in rock motion spectrum, and becomes larger when the site has a deeper or softer soil.

3) **Peak-spectrum Session:** For any given ground motion, there are a range of structures with periods between T_0 and T_s to have their resonant response to the ground motion where the mean spectrum reaches the peak. It is apparent that the peaks and valleys in the spectrum in this range vary dramatically, but remain "statically constant".T_s Is mostly in the order of 0.5 to 1 second, and is mainly dependent on the frequency contents of the ground motion. In general, the deeper or softer the soil deposit is, the larger T_s will be. This means that structures with long periods have higher potential to be subjected to peak response when located on such soil-amplified site.

4) **Flexible structure session:** For a structure with T_n > T_s, S_a tends to decay rather steadily. In practice, we only plot S_a up to T_n = 3 seconds (representing 30-40 story buildings), beyond which S_a is too small that a minimum limit has to be set for design, and wind load usually governs the design.

2.2.3. NON-LINEAR RESPONSE

The spectrum is constructed based on the assumption that the structure will remain elastic. In other words, the structure will not suffer any damage (cracking, yielding, buckling, etc) and the relation f_s = $ku(t)$ remains valid throughout the duration of the earthquake. However, most civil structures would suffer heavy damage under seismic loads so the linear spectrum would have to be modified to account for this factor. When a structure suffers damage to its members, u > u_y and once this point is reached, the stiffness becomes very small, and consequently the relationship between f_s and $u(t)$ is no longer the same. If we assume there is no increase in resisting force f_s(which is not true), the stress equation can be modified into the following:

$$f_s = \begin{cases} ku(t) \text{ when u} < u_y \ or \ \frac{df_s}{du} \neq 0 \\ f_y \text{ when } u > u_y \ or \ \frac{df_s}{du} = 0 \end{cases} \quad (2.4)$$

Where f_y represents the maximum force that can be applied to the structure in the direction of u, and is often called as yielding strength. u_y is the deformation when f_s reaches f_y and is often called yielding deformation. The following relation is obvious:

$$f_y = ku_y \quad (2.5)$$

The procedure to obtain $u(t)$ in the non linear case using Equation 2.1 together with Equations 2.2 and 2.3 is similar to what we have done to solve $u(t)$ from the elastic Equation 2.1. We can also plot $u(t)$ and find the maximum deformation D=u_{max} for each ground motion and T_n. But there are some major differences between the linear and non-linear structures and their responses, as we summarize:

1) A non linear (inelastic) system introduces additional structural properties from a linear system. In addition to stiffness, mass and damping, the non-linear system has a yielding strength and yielding deformation. It has been demonstrated from numerous response analyses that the dynamic response of the system is much more sensitive to frequency contents than a linear system.

2) In a linear(elastic) system, we only need to know the maximum deformation (D) or the maximum elastic force f_{smax}(where $f_{smax} = ku_{max} = kD = mS_a$), since elastic force and to elastic deformation are simply related on the one to one base. We can determine one by knowing the other. But this is not the case in the non-linear system, where the force and deformation are not related one to one when u_{max} > u_y. In other words, there are two variables in determining the response of a non-linear system, f_y and u_{max}.

3) Ductility in a non-linear system is used as a better parameter than u_{max}. The ductility, μ, is defined as:

$$\mu = \frac{u_{max}}{u_y} \tag{2.6}$$

Ductility can be either ductility response (or demand on the structure), or the ductility capacity, depending on what u_{max} means in the case under consideration. When u_{max} is the maximum deformation from Equation 2.1, μ corresponds to the ductility demand. If u_{max} is the maximum deformation the structure can tolerate before losing its function (partial or complete collapse), the deformation capacity; μ is defined as the ductility capacity. The ductility, a better indicator than u_{max}, is also able to indicate the level of damage that the structure suffers by providing a reference index with respect to the yielding point. Figure 9 illustrates this point.

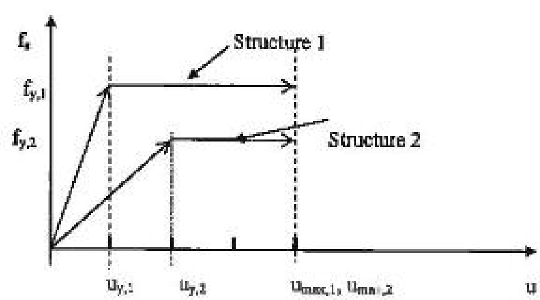

Figure 9. Levels of damage for two different structures.
Source: Chopra, Anil K. 2011.

Let's consider two structures. Structure 1 has its yielding strength and deformation $f_{y,1}$ and $u_{y,1}$, and structure 2 has yielding strength and deformation, $f_{y,2}$ and $u_{y,2}$, as shown in Figure 9. Let $u_{y,2} > u_{y,1}$. The two structures are subjected to the same ground motion, and their maximum deformation values are obtained, and indicated in Figure 9 as $u_{max,1}$ and $u_{max,2}$. If it happens that $u_{max,1}$ and $u_{max,2}$ are similar, can we say that the levels of damage are similar? The answer is no. We have to see how far the maximum deformation is from the initial yielding deformation. Even though they have the same total deformation structure one has more inelastic deformation as will suffer much heavier damage than structure 2.

2.2.4. LINEAR VS NON-LINEAR RESPONSES AND MODIFIED RESPONSE SPECTRUM

The linear spectrum is constructed based on the assumption that the structure will remain elastic, which is impossible in most cases of seismic design practice. Any structure will have a limited strength, f_y and u_y, which can be exceeded under the design earthquake. Therefore, non linear response should be considered in design. However, due to the complexity of a non linear system, direct employment of the non linear analysis is not considered practical in design. An approximate method would be sufficient. Such approximation was found by comparison between linear and non linear responses of various structures (in terms of their natural periods of vibration T_n).

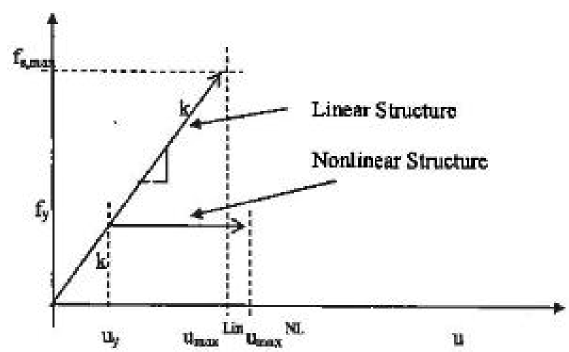

Figure 10. Linear vs non-linear behavior*. Source: Chopra, Anil K. 2011.*

As shown in Figure 10, two groups of structures: linear and nonlinear have the same stiffness's, K and natural periods of vibration T_n. The only difference between the two groups is that the non linear structure has a yielding strength of f_y while the linear structure has no such limit. These two groups of structures were subjected to an ensemble of ground motions recorded at various sites ranging from rock to soft soil deposit. The maximum displacements from response analysis between these two groups were compared, and some major observations can be summarized as follows:

1. For less rigid or more flexible structures (say those with $T_n > 0.5$sec), such as five story buildings or taller structures, the maximum displacements of linear and non linear structures are very similar.

$$u_{max}^{NL} = u_{max}^{LIN} \tag{2.7}$$

Together with the relationship,

$\mu = \frac{u_{max}}{u_y}$

We can relate f_y to $f_{s,max}$ as

$$f_{smax} = ku_{max}^{LIN}$$

$$u_{max}^{NL} = \mu u_y$$

$$f_y = ku_y$$

then

$$f_y = \frac{f_{smax}}{\mu} \qquad (2.8)$$

2. For less flexible to rigid structures with $T_n < 0.5$sec., both the linear and non linear responses are very sensitive to the types of ground motion and the relationship between the two maximum deformations is further from being equivalent. In general,

   $u_{max}^{NL} > u_{max}^{LIN}$ and $f_y > \frac{f_{smax}}{\mu}$

Equation 2.8, developed for less rigid structures, provides a simple solution to consider non-linear response of these structures by conducting linear analysis. All we need to do is use the linear response spectrum to get f_{smax}.

The maximum yielding strength f_y, then can be expressed in terms of the elastic spectrum,S_a and ductility μ, as

$$f_y = \frac{mS_a}{\mu} \qquad (2.9)$$

Equation 2.9 can be considered as a "non-linear design spectrum" that is practically used in all design codes. In fact, in design codes, S_a is first defined considering all the factors that would affect the seismic response of a structure, such as T_n, soils properties and peak ground motion values. When designing a structure, one would assume a value for ductility, which the structure is expected to have after the yielding strength has been reached.

2.3. **IBC 2009 Seismic analysis**

2.3.1. SEISMIC DESIGN CRITERIA

There are different approaches in regards to determining the seismic forces on a structure depending on its degree of irregularity, its occupancy category and its number of stories but all of them have something in common: They are based on the values of the pseudo acceleration for short periods and one second periods.

In order to determine the design pseudo acceleration values mentioned above, a number of parameters have to be calculated and different steps have to be followed. The tables and figures presented in this section can be found on the American Society of Civil Engineers 7-05(ASCE 7-05) or the International Building Code 2009 (IBC 2009).

These are the steps to determine the design acceleration parameters.

1. The first step is determining the mapped acceleration parameters. Their value assumes that the soil that seismic waves travel through is rock. Those values are well defined in America by the USGS or United States Geological Survey. They can be found in Figures 22-1 through 22-14 in the ASCE 7-05. Those parameters are S_S and S_1.

2. The next step is determining the Site Class, which determines the type of soil that there is on site so that the acceleration values can be modified to accommodate the soil in question, see Table 1 for a description of what kinds of soils corresponds to each site class.

Site Class	Soil Profile Name	Average Properties in Top 100 feet, See Section 1613.5.5		
		Soil shear wave velocity, $\bar{v}_S$, (ft/s)	Standard penetration resistance, $\bar{N}$	Soil undrained shear strength, $\bar{S}_U$, (psf)
A	Hard rock	$\bar{v}_S > 5{,}000$	N/A	N/A
B	Rock	$2{,}500 < \bar{v}_S \leq 5{,}000$	N/A	N/A
C	Very dense soil and soft rock	$1{,}200 < \bar{v}_S \leq 2{,}500$	$\bar{N} > 50$	$\bar{S}_U > 2{,}000$
D	Stiff soil profile	$600 \leq \bar{v}_S \leq 1{,}200$	$15 \leq \bar{N} \leq 50$	$1{,}000 \leq \bar{S}_U \leq 2{,}000$
E	Soft soil profile	$\bar{v}_S < 600$	$\bar{N} < 15$	$\bar{S}_U < 1{,}000$
E	---	Any profile with more than 10 feet of soil having the following characteristics: 1. Plasticity index PI>20, 2. Moisture content w≥40%, and 3. Undrained shear strength $\bar{S}_U < 500$ psf		
F	---	Any profile containing soils having one or more of the following characteristics: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. 2. Peats and/or highly organic clays (H>10 feet of peat and/or highly organic clay where H = thickness of soil) 3. Very high plasticity clays (H>25 feet with plasticity index PI>75) 4. Very thick soft/medium stiff clays (H>120 feet)		

Table 1. Site class definition. *Source: IBC 09*

Once the site class is determined, the site coefficients F_a and F_v can be determined too. See Tables 2 and 3.

Site Class	Mapped Spectral Response Acceleration at Short Period				
	$S_S \leq 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	Note b				

a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at short periods, S_S.

b. Values shall be determined in accordance with Section 11.4.7 of ASCE 7.

Table 2. Site coefficient Fa computation. *Source: IBC 09*

Site Class	Mapped Spectral Response Acceleration at 1-Second Period				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	Note b				

a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at 1-second period, S_1.

b. Values shall be determined in accordance with Section 11.4.7 of ASCE 7.

Table 3. Site coefficient Fv computation. *Source: IBC 09*

3. The site coefficients just computed will allow to determine the Adjusted Maximum Considered Earthquake (MCE).

$$S_{MS} = F_a S_s \qquad (2.10)$$

$$S_{M1} = F_V S_1 \qquad (2.11)$$

4. The next step is determining the Design Spectral Acceleration Parameters, which are given by the following formulas:

$$S_{DS} = 2/3 S_{MS} \qquad (2.12)$$

$$S_{D1} = 2/3 S_{M1} \qquad (2.13)$$

Having defined the design spectral acceleration values leads to generating the Design Response Spectrum. See Figure 11.

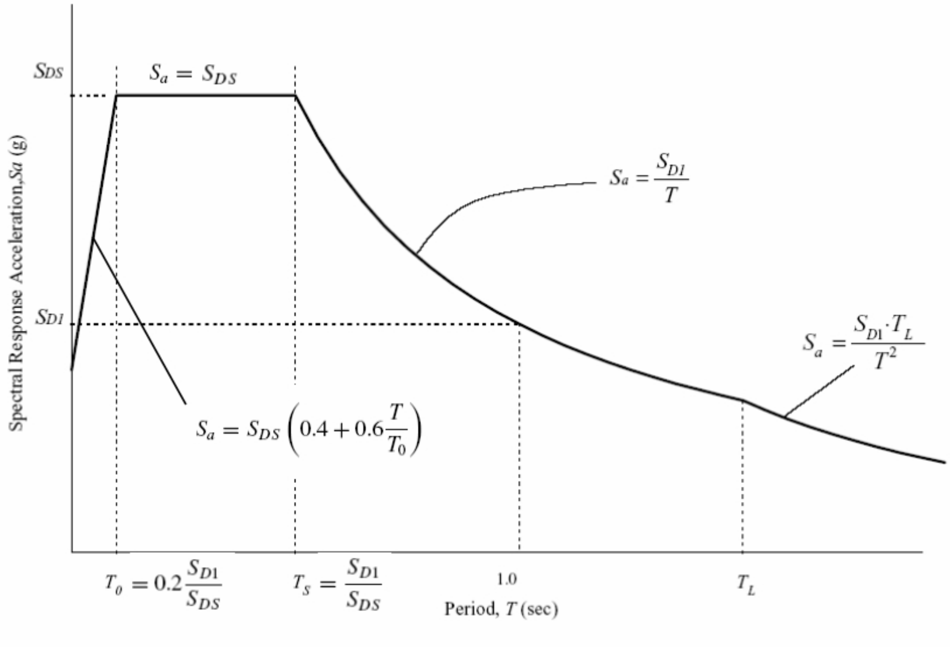

Figure 11. Design response spectrum. *Source: ASCE 7-05*

Where site-specific ground motion procedures are not used, the design response spectrum may be used and it is developed as follows:

1. For periods less than T_0, the design spectral response acceleration S_a shall be as given in Equation 2.14.

$$S_a = S_{DS}\ \ 0.4 + 0.6\frac{T}{T_0} \tag{2.14}$$

2. For periods greater than or equal to T_0 and less than or equal to T_S, the design spectral response S_a, shall be taken equal to S_{DS}.

3. For periods greater than T_S, and less than or equal to T_L, the design spectral response acceleration, S_a, shall be taken as given by Equation 2.15.

$$S_a = \frac{S_{D1}}{T} \tag{2.15}$$

4. For periods greater than T_L, S_a shall be taken as given by Equation 2.16.

$$S_a = \frac{S_{D1}T_L}{T} \tag{2.16}$$

Where:

T = the fundamental period of the structure

$T_0 = 0.2\frac{S_{D1}}{S_{DS}}$ (2.17)

$T_S = \frac{S_{D1}}{S_{DS}}$ (2.18)

T_L = Long period transition period. Its value can be mapped and its value ranges from 6 to 16 seconds.

Now we can determine the acceleration that our structure will have at a specific site as long as we can determine its natural period of vibration in a somewhat accurate way. The ASCE 7-05 presents approximate methods of calculating the natural period of a structure.

The fundamental period of vibration in the direction under consideration shall be determined using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis.
This process can be very tedious depending on all of the properties mentioned above and the ASCE presents a simplified method of computing the fundamental period of vibration of any building and it is called the "Approximate fundamental period method" and it will be explained in the following paragraph.

The approximate fundamental period (T_a), in seconds, shall be determined from the following equation:

$T_a = C_t h_n^x$ (2.19)

Whereh_n is the height in feet above the base to the highest level of the structure and the coefficients C_t and x are determined from table 12.8-2 (Table 4) of the ASCE 7-05 shown below.

TABLE 12.8-2 VALUES OF APPROXIMATE PERIOD PARAMETERS C_t AND x

Structure Type	C_t	x
Moment-resisting frame systems in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by components that are more rigid and will prevent the frames from deflecting where subjected to seismic forces:		
Steel moment-resisting frames	0.028 (0.0724)[a]	0.8
Concrete moment-resisting frames	0.016 (0.0466)[a]	0.9
Eccentrically braced steel frames	0.03 (0.0731)[a]	0.75
All other structural systems	0.02 (0.0488)[a]	0.75

[a] Metric equivalents are shown in parentheses.

Table 4. Values of approximate period parameters.
Source: ASCE 7-05

What we exposed was the seismic design criteria and the computation of the most significant parameters, but have not provided yet a method to compute an equivalent lateral force applied to a structure. The method most typically used for regularly shaped and non-critical structures less than 20 stories high is the Equivalent Lateral Force Procedure, which will be exposed, in the following paragraph:

2.3.2. EQUIVALENT LATERAL FORCE PROCEDURE METHOD

The purpose of any analysis procedure is to come up with an equivalent lateral force and distribution along its geometry. The Equivalent Lateral Force Method (ELFM) is directly based on the design response spectrum. The equivalent seismic shear is calculated as follows:

$$V = C_s W \tag{2.20}$$

Where $C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)}$ (2.21)

R is the response modification factor and takes into consideration the design ductility that we want our structure to have. Higher values of R will reduce our design force and therefore, will make our structure yield earlier and absorb more plastic deformation. The value of R can be determined from Table 12.2.1 of the ASCE 7-05. This table will not be shown here due to its size but will partially be shown on the case study when a value for R has to be determined.

I is the importance factor and it adjusts the value of our equivalent seismic force to the importance that our structure has. For more critical structures or structures containing hazardous materials higher values of I are going to be used, therefore reducing the inelasticity of the structure and reducing the potential for damage. Determining the importance of the structure comes down to locating the structure in question in table 1-1 of the ASCE 7-05(Table 6) shown on the next page. Once the occupancy category has been determined the value of I can be determined through table 11.5-1(Table 5).

TABLE 11.5-1 IMPORTANCE FACTORS

Occupancy Category	*I*
I or II	1.0
III	1.25
IV	1.5

Table 5. Importance factors. *Source: ASCE 7-05*

OCCUPANCY CATEGORY	NATURE OF OCCUPANCY
I	Buildings and other structures that represent a low hazard to human life in the event of failure, including but not limited to: • Agricultural facilities. • Certain temporary facilities. • Minor storage facilities.
II	Buildings and other structures except those listed in Occupancy Categories I, III and IV
III	Buildings and other structures that represent a substantial hazard to human life in the event of failure, including but not limited to: • Covered structures whose primary occupancy is public assembly with an occupant load greater than 300. • Buildings and other structures with elementary school, secondary school or day care facilities with an occupant load greater than 250. • Buildings and other structures with an occupant load greater than 500 for colleges or adult education facilities. • Health care facilities with an occupant load of 50 or more resident patients, but not having surgery or emergency treatment facilities. • Jails and detention facilities. • Any other occupancy with an occupant load greater than 5,000. • Power-generating stations, water treatment for potable water, waste water treatment facilities and other public utility facilities not included in Occupancy Category IV. • Buildings and other structures not included in Occupancy Category IV containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released.
IV	Buildings and other structures designated as essential facilities, including but not limited to: • Hospitals and other health care facilities having surgery or emergency treatment facilities. • Fire, rescue and police stations and emergency vehicle garages. • Designated earthquake, hurricane or other emergency shelters. • Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response. • Power-generating stations and other public utility facilities required as emergency backup facilities for Occupancy Category IV structures. • Structures containing highly toxic materials as defined by Section 307 where the quantity of the material exceeds the maximum allowable quantities of Table 307.1.(2). • Aviation control towers, air traffic control centers and emergency aircraft hangars. • Buildings and other structures having critical national defense functions. • Water treatment facilities required to maintain water pressure for fire suppression.

Table 6. Occupancy Category of buildings. *Source: ASCE 7-05*

Therefore, C_S corresponds to the maximum design acceleration that a structure would have taking into consideration how critical it is and the ductility we want it to have.

The value of C_S shall not exceed the following:

1. $C_S = \frac{S_{D1}}{T(\frac{R}{I})}$ for $T \leq T_L$ (2.22)

2. $C_S = \frac{S_{D1}T_L}{T(\frac{R}{I})}$ for $T \geq T_L$ (2.23)

and,

C_SShall not be less than 0.01.

The value of C_S and the limitations in it come from the design spectrum that we previously computed, where the maximum value would be the modified acceleration at short periods.

This section explained the theory behind seismic forces and provided a method for determining the seismic forces given a structure and a location. An example on how to use this method will be given in this thesis for a single story wood framed building.

3 **WOOD ENGINEERING**

3.1. **Sawn lumber**

3.1.1. INTRODUCTION

The terms sawn lumber and solid sawn lumber are often used to refer to wood members that have been manufactured by cutting a member directly from a log. Other structural members may start as lumber, which then undergoes additional fabrication processes. For example, small pieces of lumber can be graded into laminating stock then glued and laid up to form larger wood members, known as glued laminated timbers, or Glulams.
Many other wood based products are available for use in structural applications. Some examples include solid members such as wood poles and timber piles, fabricated components such as trusses, wood I joists, and box beams. A number of these products are recent developments in the wood industry. They are the result of new technology and the economic need to make use of different species and smaller trees that cannot be used to produce sawn lumber.

The scope of this thesis is to introduce many of the important physical and mechanical properties of wood in addition to the sizes and existing grades of sawn lumber.

In America, the NDS or the National Design Standard, which pertains to the American Wood Council, is the code that governs the design in wood construction and will be frequently mentioned in this section and throughout the remainder of the thesis.

3.1.2. CELLULAR MAKE UP

As a biological material, wood represents a unique structural material because growing new trees in forests, which have been harvested, can renew its supply. Proper forest management is necessary to ensure a continuing supply of lumber.

Wood is composed of elongated, round, or rectangular tube like cells as can be seen in Figure 12. These cells are much longer than they are wide, and the length of the cells is essentially parallel to the length of the tree. The cell walls are made up of cellulose, and the cells are bound together by material known as lignin.

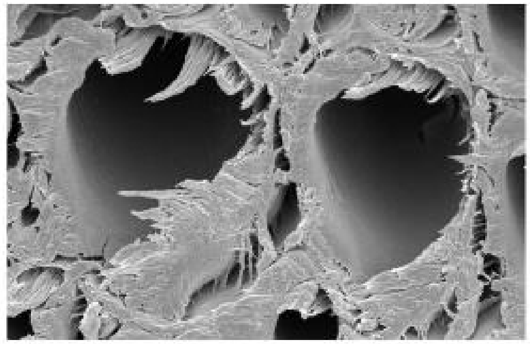

Figure 12. Cells bounded together by lignin*. Source: Aghayere, Abi. 2007.*

If the cross section of a log is examined, concentric rings are seen. One ring represents the amount of wood material, which is deposited, on the outside of the tree during one growing season. One ring then is termed as annual ring.

The annual rings develop because of the differences in the wood cells that are formed in the early portion of the growing season compared with those formed toward the end of the growing season. Large, thin-walled cells are formed at the beginning of the growing season. These are known as early-wood or spring-wood cells. The cells deposited on the outside of the annual ring toward the end of the growing season are smaller, have thicker walls, and are known as latewood or summerwood cells. It should be noted that the annual rings occur only in trees that are located in climate zones, which have distinct growing seasons. In tropical zones, trees produce wood cells, which are uniform throughout the entire year.

Because summerwood is denser than springwood, it is stronger. The annual rings, therefore, provide one of the visual means by which the strength of a piece of wood may be evaluated. The more summerwood in relation to the amount of springwood, the stronger the piece of lumber. This comparison is normally made by counting the number of growth rings per unit width of cross section. See Figure 13 for illustration of the annual ring structure.

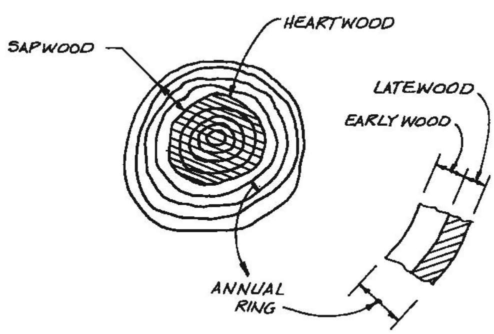

Figure 13. Anual ring structure. *Source: Breyer, Donald E. 1999.*

In addition to the annual rings, two different colors of wood may be noticed in the cross section of the log. The darker center portion of the log is known as heartwood. The lighter portion of the wood near the exterior of the log is known as sapwood. The relative amount of heartwood compared with sapwood varies with the species of tree. Heartwood, because it occurs at the center of the tree, is obviously much older than sapwood, and, in fact, heartwood represents the wood cells that are inactive. These cells, however, provide strength and support to the tree. Sapwood is used to store food and transport water. The strength of heartwood and sapwood is essentially the same. Heartwood is somewhat more decay-resistant than sapwood, but sapwood more readily accepts penetration by wood-preserving chemicals

3.1.3. MOISTURE CONTENT AND SHRINKAGE

The solid portion of wood is made of a complex cellulose-lignin compound. The cellulose comprises the framework of the cell walls, and the lignin cements and binds the cells together.

In addition to the solid material, wood contains moisture. The moisture content (MC) is measured as a percentage of water to the oven dry weight of a piece of wood. The moisture content in a living tree can be as high as 200 percent. However, the moisture content of structural lumber in service is much less. The average moisture content that lumber assumes in service is known as the equilibrium moisture content (EMC). The EMC will range somewhere between 7 and 14 percent.

Moisture is held within wood in two ways. Water contained in the cell cavity is known as free water. As wood dries, the first water to be driven off is the free water. The moisture content that corresponds to a complete loss of free water is known as fiber saturation point (FSP). No volume changes or changes in other structural properties are associated with changes in moisture content above the fiber saturation point. See Figure 14 for an illustration of the most representative moisture contents in a piece of wood.
However, with moisture content changes below the fiber saturation point, bound water is lost and volume changes occur. If moisture is lost, wood shrinks: if moisture is gained, wood swells. Decreases in moisture content below the fiber saturation point are accompanied by increases in strength properties. Test results have shown that strength properties peak at around 10 to 15 percent MC.

Figure 14 shows the moisture content in lumber in comparison with its solid weight. The values indicate that the lumber was manufactured (point 1) at a MC below the fiber saturation point. Some additional drying occurred before the lumber was used in construction (point 2). The EMC is shown to be less than the MC at the time of construction. That is typical for most buildings.

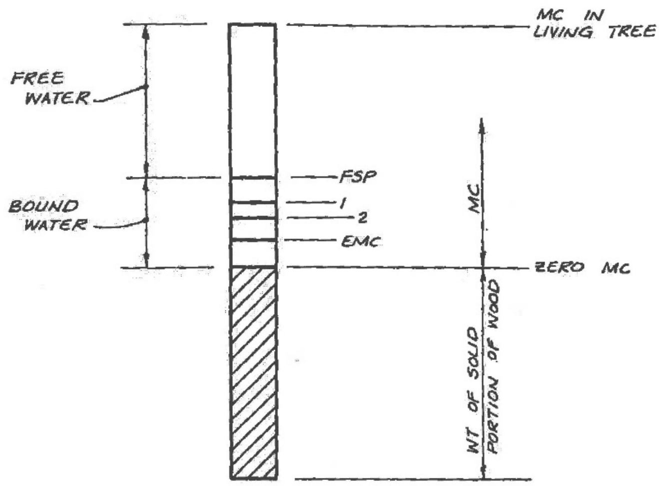

Figure 14. Moisture contents in a piece of wood.*Source: Breyer, Donald E. 1999*

3.1.4. DURABILITY OF WOOD AND THE NEED FOR PRESSURE TREATMENT

The discussion of the moisture content of lumber often leads to concerns about the durability of wood structures and the potential for decay. However, the record is clear. If wood is used properly, it can be a permanent building material. If wood is used incorrectly, major problems can develop, sometimes very rapidly. Understanding the material is the key to its proper use. The performance of classic wood structures is testimony to the durability of wood in properly designed structures.

Generally, if wood is protected and used at relatively low moisture content, wood performs satisfactorily without chemical treatment.
High moisture content values can occur in wood roof systems over swimming pools and in processing plants with high humidity conditions. High moisture is defined as exceeding 19 percent in sawn lumber and 16 percent in Glulams. Problems involving high moisture content can also occur in geographic locations with high humidity.
When required for new construction, chemicals can be impregnated into the lumber and other wood products by a pressure treating process. The chemical preservatives prevent or effectively retard the destruction of wood.
Many species, especially the southern pine, readily accept preservative treatments. Other species, however, do not accept pressure treatments as well and require incising to make the treatment effective. In effect, incised lumber has small cuts, or incisions, made into tall four sides along its length. The incisions create more surface area for the chemicals to penetrate the wood member, thereby increasing the effectiveness of the pressure treating. Rather than focusing only on the moisture content, a more complete overview of the question of long-term performance and durability recognizes that several instruments can destroy wood. The major ones are:

1. Decay
2. Termites
3. Marine borers
4. Fire

1. Decay is caused by fungi which feed on the cellulose or lignin of the wood. These fungi must have food, moisture, air and favorable temperatures. All of these items are required for decay to occur. If any of the requirements is not present, decay will not occur. Thus, untreated wood that is continuously dry will not decay. Exposure to weather can set up the conditions necessary for wood to decay. Pressure treatment introduces chemicals that poison the food supply of the fungi.

2. Termites can be found in most areas of the United States, but they are more of a problem in the warmer-climate areas. Subterranean termites are the most common, but drywood and dampwood species also exist. Subterranean termites nest in the ground and enter wood which is near or in contact with damp ground. The cellulose forms the food supply for the termites.

3. Marine borers are found in salty waters, and they present a problem in the design of marine piles. Pressure treated piles have an extensive records in resisting attack by marine borers.

4. Where necessary to meet building code requirements, or where the designer decides that an extra measure of fire protection is desirable, fire retardant wood may be used. This type of treatment involves the use of chemicals in formulations that have fire-retardant properties. Some of the type of chemicals used are preservatives and thus also provide termite protection. Fire retardant treatment, however, requires higher concentrations of chemicals in the treated zone than normal preservative treatments. In regards to that treatment reducing the allowable stress of wood, preservative treatments do not modify wood strength properties with one exception: if the lumber is incised. Incising effectively decreases the strength and stiffness and must be accounted for in design when incised lumber is used, as explained in following sections of this thesis. It is important to note that the reduction in stress depends on the type of process used and for instance, the NDS refers the designer to the company providing the fire retardant treatment for the appropriate factors.

3.1.5. METHODS OF GRADING LUMBER

The majority of sawn lumber is graded by visual inspection, and material graded in this way is known as visually graded structural lumber. As the lumber comes out of the mill, a person familiar with the lumber grading rules examines each piece and assigns a grade by stamping the member. The grade stamp includes the grade, the species or species group, and other pertinent information. If the lumber grade has recognized mechanical properties for use in structural design, it is referred to as a stress grade.

The lumber grading rules establish the limits on the size and number of growth characteristics that are permitted in the various stress grades.

It is important to consider that more than one set of grading rules can be used to grade some commercial species groups. For example, Douglas Fir Larch can be graded under Western Wood Products Association (WWPA) rules or under West Coast Lumber Inspection Bureau (WCLIB) rules. There are some differences in allowable stresses between the two sets of rules. The tables of design properties in the NDS supplement have the grading rules clearly identified. The differences in allowable stresses occur only in large-size members known as *Timbers*, and allowable stresses are the same under both sets of grading rules for *Dimension Lumber.* See Figure 15.

Figure 15. Visually graded lumber stamp*. Source: Breyer, Donald E. 1999*

Figure 15 indicates the following:

1. Lumber grading agency: WWPA
2. Mill number: 12
3. Lumber grade: Select Structural
4. Commercial lumber species: Douglas Fir-Larch
5. Moisture content at the time of surfacing: S-GRN

Although most lumber is visually graded, a small percentage of lumber is machine stress rated by subjecting each piece of wood to a nondestructive test. The nondestructive test is highly automated, and the process takes very little time. As lumber comes out of the mill, it passes through a series of rollers. In this process, a bending load is applied about the minor axis of the cross section, and the modulus of elasticity of each piece is measured. In addition to the nondestructive test, machine stress rated lumber is subjected to s visual check. Because of the testing procedures, machine stress rating is limited to thin material (2 inches). Lumber graded in this manner is known as MSR lumber.
Each piece of MSR lumber is stamped with a grade stamp that allows it to be fully identified, and the grade stamp for MSR lumber differs from the stamp for visually graded lumber. The grade stamp for MSR lumber includes a numerical value of nominal bending stress and modulus of elasticity. See Figure 16.

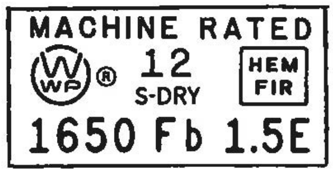

Figure 16. Mechanical graded lumber stamp.*Source: Breyer, Donald E. 1999*

Figure 16 indicates the following:

1. MSR marking: Machine Rated
2. Lumber grading agency: WWPA
3. Mill number: 12
4. Nominal bending stress and modulus of elasticity: 1,650psi and 1.5x10^6psi
5. Commercial lumber species: Hem-Fir
6. Moisture at time of surfacing: S-DRY

3.1.6. SPECIES AND SPECIES GROUPS

A large number of species of trees can be used to produce structural lumber. As a general rule, a number of species are grown, harvested, manufactured, and marketed together. From a practical standpoint, the structural designer uses lumber from a commercial species group rather than a specific individual species. The same grading rules, tabulated stresses, and a grade stamps are applied to all species in the species group. Tabulated stresses for a species group were derived using statistical procedures that ensure conservative values for all species in the group.

In some cases, the mark of one or more species from a species group is identified in the grade stamp. When one or more species from a species group are identified in the grade stamp, the allowable stresses for the species group are the appropriate stresses for use in structural design. Special knowledge in wood identification would be required to determine the individual species.

The 2005 NDS supplement contains a complete list of individual species groups along with a summary of the various individual species of trees that may be included in each group. See Figure 17. It is important to understand that each species group is assigned different stress values.

The choice of species for use in design is typically a matter of economics. For a given location, only a few species groups will be available, and a check with local lumber distributors or a wood products agency will narrow the selection down considerably.

Species Group Name and *Group Name Mark* that may appear in grade stamp	*Individual Species* that may be included in the Species Group and *Individual Species Mark* that may appear in grade stamp	Notes
Douglas Fir-Larch DOUG. FIR-L	Douglas Fir D FIR Western Larch	Individual species mark for Douglas Fir may also appear as "DOUGLAS FIR" or "D. FIR"
Douglas Fir-Larch (N) D. FIR(N)	Douglas Fir[1] Western Larch[1]	(N) - indicates a Canadian species group
Douglas Fir South D FIR S	Douglas Fir South D FIR S	South indicates Douglas Fir grown in Arizona, Colorado, Nevada, New Mexico, and Utah
Hem-Fir HEM FIR	California Red Fir[1] Grand-Fir[1] Nobel Fir[1] Pacific Silver Fir[1] Western Hemlock HEM White Fir[1]	
Hem-Fir (N) HEM-FIR-N	Amabilis Fir Western Hemlock	(N) - indicates a Canadian species group
Southern Pine SYP	Loblolly Pine[1] Longleaf Pine[1] Shortleaf Pine[1] Slash Pine[1]	Group mark is not used when graded under Southern Pine Inspection Bureau - grade stamp will show: SPIB
Spruce-Pine-Fir S-P-F	Alpine Fir[1] Balsam Fir[1] Black Spruce[1] Englemann Spruce[1] Jack Pine[1] Lodgepole Pine[1] Red Spruce[1] White Spruce[1]	Canadian species group
Spruce-Pine-Fir (S) SPF^S	Balsam Fir[2] Eastern Spruce[2] Engelmann Spruce ES Jack Pine[1] Lodgepole Pine LP Red Pine[2] Sitka Spruce SS	(S) - indicates USA species group (established 1991). Eastern Spruce is any combination of Black Spruce, Red Spruce, and White Spruce.

Figure 17. Species and group species. *Source: Breyer, Donald E. 1999*

3.1.7. SIZES OF STRUCTURAL LUMBER

Structural calculations are based on the standard net size of a piece of lumber. The designer may have to allow for shrinkage when detailing connections, but standard dimensions are accepted for stress calculations.

Most structural lumber is dressed lumber. In other words, the lumber is surfaced to the standard net size, which is less than the nominal (stated) size. Lumber is dressed on a planning machine for the purpose of obtaining smooth surfaced and uniform sizes.

Dressed lumber is used in many structural applications, but large timbers are commonly rough sawn to dimensions that are close to the standard net sizes. The textured surface of rough-sawn lumber may be desired for architectural purposes and may be specially ordered in smaller pieces.

The cross sectional dimensions of rough sawn lumber are approximately 1/8 in larger than the standard dressed size.

See Figure 18 for an illustration of all the different sizes in wood.

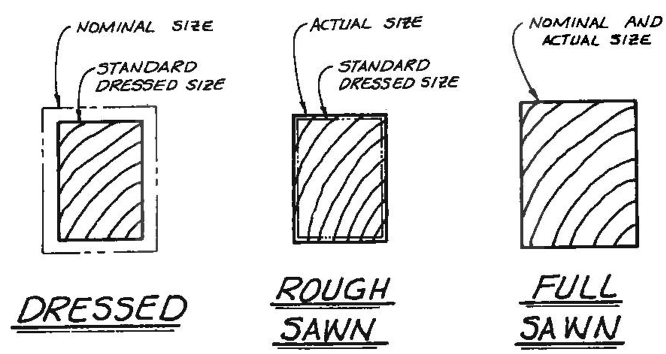

Figure 18. Lumber sizes.*Source: Breyer, Donald E. 1999*

3.1.8. SIZE CATEGORIES AND STRESS GRADES

Sawn lumber from a log, regardless of species and size, is quite variable in mechanical properties. Pieces may differ in strength by several hundred percent. For simplicity and economy in use, pieces of wood of similar mechanical properties are placed in categories called stress grades, which are characterized by (a) one or more sorting criteria, (b) a set of properties for engineering design, and (c) a unique grade name. The most familiar system is that for lumber.

The lumber grading rules, which establish allowable stresses for use in structural design, have been developed over many years. In this development process, the relative size of a piece of wood was used as a guide for anticipating the application or "use" that a member would receive in the field. For example, pieces of lumber with rectangular cross sections make more efficient beams than members with square cross sections. Thus, if the final application of a piece of wood were known, the stress grading rules would take into consideration the primary function of the member(axial strength or bending strength). See Figure 19.

Boards	**1 to 1½ in. thick**
	2 in. and wider
Dimension lumber	**2 to 4 in. thick**
	2 in. and wider
Timbers	**5 in. and thicker**
	5 in. and wider

Figure 19. Size categories.*Source: Breyer, Donald E. 1999*

The "National Grading Rule" establishes the lumber classifications and grade names for visually stress-graded dimension lumber and can be seen in Table 7. The strength ratio is the hypothetical ratio of the strength of a piece of lumber with visible strength-reducing growth characteristics to its strength if those were absent. Design properties will vary by size, species and grade and are published in the appropriate rule books and in the National Design Specification for Wood Construction (NDS).

Lumber classification[a]	Grade name	Bending strength ratio (%)
Light framing[b]	Construction	34
	Standard	19
	Utility	9
Structural light framing[b]	Select Structural	67
	1	55
	2	45
	3	26
Stud[c]	Stud	26
Structural joists and planks[d]	Select Structural	65
	1	55
	2	45
	3	26

[a]Contact rules-writing agencies for additional information.
[b]Standard 38 to 89 mm (nominal 2 to 4 in.) thick and wide. Widths narrower than 89 mm (4 in. nominal) may have different strength ratio than shown.
[c]Standard 38 to 89 mm (nominal 2 to 4 in.) thick, ≥38 mm (≥4 in. nominal) wide.
[d]Standard 38 to 89 mm (nominal 2 to 4 in.) thick, ≥114 mm (≥5 in. nominal) wide.

Table 7. Stress grades*. Source: David E. Kretschmann. 2010.*

It has been noted that size and use are related. However, in the process of determining the allowable stresses for a member, the structural designer needs to place emphasis on understanding the size classifications. The reason is that different allowable stresses apply to the same stress grade name in the different size categories. Allowable stresses for a given commercial species of lumber are generally different for “Select Structural” in all of the size categories. See Table 8.This means that the wider the cross section gets the smaller the values for allowable stresses are.

A couple of important points should be made about the size and use categories.

1. Decking is normally stressed in bending about the minor axis of the cross section and allowable stresses for decking are listed in a separate table.

2. Allowable stresses for dimension lumber are given in a number of separate tables. In these tables the stress grades are grouped together regardless of the size.

USE WITH TABLE 4B ADJUSTMENT FACTORS

Species and commercial grade	Size classification	Design values in pounds per square inch (psi)						
		Bending F_b	Tension parallel to grain F_t	Shear parallel to grain F_v	Compression perpendicular to grain $F_{c\perp}$	Compression parallel to grain F_c	Modulus of Elasticity E	E_{min}
SOUTHERN PINE								
Dense Select Structural	2" - 4" wide	3,050	1,650	175	660	2,250	1,900,000	690,000
Select Structural		2,850	1,600	175	565	2,100	1,800,000	660,000
Non-Dense Select Structural		2,650	1,350	175	480	1,950	1,700,000	620,000
No.1 Dense		2,000	1,100	175	660	2,000	1,800,000	660,000
No.1		1,850	1,050	175	565	1,850	1,700,000	620,000
No.1 Non-Dense		1,700	900	175	480	1,700	1,600,000	580,000
No.2 Dense		1,700	875	175	660	1,850	1,700,000	620,000
No.2		1,500	825	175	565	1,650	1,600,000	580,000
No.2 Non-Dense		1,350	775	175	480	1,600	1,400,000	510,000
No.3 and Stud		850	475	175	565	975	1,400,000	510,000
Construction	4" wide	1,100	625	175	565	1,800	1,500,000	550,000
Standard		625	350	175	565	1,500	1,300,000	470,000
Utility		300	175	175	565	975	1,300,000	470,000
Dense Select Structural	5" - 6" wide	2,700	1,500	175	660	2,150	1,900,000	690,000
Select Structural		2,550	1,400	175	565	2,000	1,800,000	660,000
Non-Dense Select Structural		2,350	1,200	175	480	1,850	1,700,000	620,000
No.1 Dense		1,750	950	175	660	1,900	1,800,000	660,000
No.1		1,650	900	175	565	1,750	1,700,000	620,000
No.1 Non-Dense		1,500	800	175	480	1,600	1,600,000	580,000
No.2 Dense		1,450	775	175	660	1,750	1,700,000	620,000
No.2		1,250	725	175	565	1,600	1,600,000	580,000
No.2 Non-Dense		1,150	675	175	480	1,500	1,400,000	510,000
No.3 and Stud		750	425	175	565	925	1,400,000	510,000
Dense Select Structural	8" wide	2,450	1,350	175	660	2,050	1,900,000	690,000
Select Structural		2,300	1,300	175	565	1,900	1,800,000	660,000
Non-Dense Select Structural		2,100	1,100	175	480	1,750	1,700,000	620,000
No.1 Dense		1,650	875	175	660	1,800	1,800,000	660,000
No.1		1,500	825	175	565	1,650	1,700,000	620,000
No.1 Non-Dense		1,350	725	175	480	1,550	1,600,000	580,000
No.2 Dense		1,400	675	175	660	1,700	1,700,000	620,000
No.2		1,200	650	175	565	1,550	1,600,000	580,000
No.2 Non-Dense		1,100	600	175	480	1,450	1,400,000	510,000
No.3 and Stud		700	400	175	565	875	1,400,000	510,000

Table 8. Stress grade values for different size categories for Southern Pine. *Source: NDS 2005 Supplement .*

3.1.9. ADJUSTMENT FACTORS IN WOOD DESIGN

Since wood is a material with unique properties, its proper use may require a number of adjustment factors. Although the basic concepts of timber design are very straightforward, the many possible adjustment factors can make wood design cumbersome at the beginning.

Generally speaking, the forces and stresses in wood structures are computed according to the principles of engineering mechanics and strength of materials. The same basic linear elastic theory is applied in the design of wood beams as is applied to the design of concrete or steel members. The unique properties of wood members and the differences in behavior are usually taken into account with adjustment factors.

For consistency, it is highly recommended that the adjustment factors for wood design be kept as multiplying factors for stresses.

Allowable stresses

Tabulated stresses for wood simply represent a starting point in the determination of the allowable stress for a particular design. Allowable stresses are found by multiplying the tabulated stresses by the appropriate adjustment factors.
For a design to be acceptable, the actual stress must be less than or equal to the allowable stress.

The NDS provides a list of all the different allowable stresses to be used with the corresponding adjustment factors in its supplement. The stresses provided are the bending stress, the stress for tension parallel to grain, shear parallel to grain, and compression parallel and perpendicular to grain along with the modulus of elasticity. This can be seen in Table 9.

It is also important to note that not all the adjustment factors apply to all the different stresses, some of them might apply only to certain stresses.

		ASD Only	ASD and LRFD										LRFD Only		
		Load Duration Factor	Wet Service Factor	Temperature Factor	Beam Stability Factor	Size Factor	Flat Use Factor	Incising Factor	Repetitive Member Factor	Column Stability Factor	Buckling Stiffness Factor	Bearing Area Factor	Format Conversion Factor	Resistance Factor	Time Effect Factor
$F_b' = F_b$	×	C_D	C_M	C_t	C_L	C_F	C_{fu}	C_i	C_r	-	-	-	K_F	φ_b	λ
$F_t' = F_t$	×	C_D	C_M	C_t	-	C_F	-	C_i	-	-	-	-	K_F	φ_t	λ
$F_v' = F_v$	×	C_D	C_M	C_t	-	-	-	C_i	-	-	-	-	K_F	φ_v	λ
$F_{c\perp}' = F_{c\perp}$	×	-	C_M	C_t	-	-	-	C_i	-	-	-	C_b	K_F	φ_c	λ
$F_c' = F_c$	×	C_D	C_M	C_t	-	C_F	-	C_i	-	C_P	-	-	K_F	φ_c	λ
$E' = E$	×	-	C_M	C_t	-	-	-	C_i	-	-	-	-	-	-	-
$E_{min}' = E_{min}$	×	-	C_M	C_t	-	-	-	C_i	-	-	C_T	-	K_F	φ_s	-

Table 9. Adjustment factors in sawn lumber. *Source: NDS 2005.*

The adjustment factors in wood design are usually given the symbol of an uppercase C, and one or more subscripts added to indicate the purpose of the adjustment. Some of the subscripts are uppercase letters, and others are lowercase. There are a large number of adjustment factors and going through them one by one is not part of the scope of this thesis. Nevertheless the most important and representative adjustment factors will be explained in this section.

These are some of the adjustment factors used in wood design.

1 C_M= wet service factor

2 C_D = load duraction factor

3 C_F= Size factor

4 C_{fu} = flat use factor

5 C_i = Incising factor

6 C_T = Temperature factor

7 C_r = Repetitive member factor

8 C_H = Shear stress factor

9 C_L = Beam stability factor

These adjustment factors do not apply to all tabulated values. In addition, other adjustments may be necessary in certain types of problems. For example, the column stability factor C_P is required in the design of wood columns.

The large number of factors is an attempt to remind the designer to not overlook something that can affect the performance of a structure. However, in many practical design situations, a large number of adjustment factors may have a value of 1. In such a case, the adjustment is said to default to unity. Thus, in many common designs, the problem will not be as complex as the long list of adjustment factors would make it appear.

1. Wet Service Factor C_M

The moisture content of wood and its relationship to strength were previously described in this thesis already. Generally, for a moisture content of 19% or less, the wet service factor will be 1.0. For moisture contents exceeding 19%, the wet service factor will be less than 1.0.

2. Load Duration Factor C_D

Wood has a unique structural property. It can support higher stresses if the loads are applied for a short period of time. This is particularly significant when one realizes that if an overload occurs, it is probably the result of a temporary load. All tabulated design values for connections apply to normal duration loading.
The load duration factor is the adjustment factor used to convert tabulated stresses and nominal fastener values to allowable values based on the expected duration of full design load.
Normal duration is taken as 10 years, and floor live loads are conservatively associated with this time of loading. For other loads, the duration factor lies in the range 0.9<C_D<2.0. The maximum value for the load duration factor comes from the shortest type of load which corresponds to the impact load.

3. Size factor C_F

It has been known for some time that the size of a wood member has an effect on its unit strength (stress). This behavior is taken into account through the size factor C_F.
The size factors for most species of visually graded dimension lumber are summarized in the Adjustment factor section of the code in question. The size factors depend on the

stress grade and the width of the piece of lumber, for bending stress, the thickness also affects the size factor.

4. Flat use factor C_{fu}

Except for decking, tabulated bending stresses for dimension lumber apply to wood members that are stressed in flexure about their strong axis. In a limited number of situations, dimension lumber may be loaded in bending about the minor axis of the cross section. The term flat describes this application, where the load is applied to the flat face of the member.

5. Incising factor C_i

Many wood species, most notably southern pines, readily accept preservative treatments, while other species do not accept pressure treatments as well. For species that are not easily treated, incising is often used to make the treatment effective. If incising is used to increase the penetration of the preservatives, some design tabulated values must be adjusted. For the modulus of elasticity (C_i= 0.95), and for bending stress, tension stress and compression parallel to grain (C_i=0.85). The incising factor is taken as one for all other design values.

6. Temperature factor C_T

The strength of a member is affected by the temperature of wood in service. Strength is increased as the temperature cools below the normal temperature range found in most buildings. On the other hand, the strength decreases as temperatures are increased. The temperature factor C_T is the multiplier that is used to multiply the tabulated stresses if prolonged exposure to higher than normal temperatures are encountered in a design situation.

7. Repetitive member factor C_r

Many wood structures have a series of closely spaced parallel members. The members are often connected together by sheathing or decking. In this arrangement, the performance of the system does not depend solely on the capacity of an individual member. This can be contrasted to an engineered wood structure with relatively large structural members spaced a greater distance apart. The failure of one large member would essentially be a failure of the entire system. The system performance of a series of small, closely spaced members is recognized by the NDS by providing a 15% increase in the tabulated bending stress values. It only applies to the bending stress values for dimension lumber used in a repetitive system, which is defined as one that has:

1. Three or more parallel members of dimension lumber.

2. Members spaced mot more than 24 inches.

3. Members connected together by a load-distributing element such as roof, floor or wall sheathing.

8. Shear stress factor C_H

The presence of splits, checks, and shakes reduced the flexural capacity of a wood member.
Tabulated values of the shear stress F_V for sawn lumber reflect the assumption that the member may be split along its full length. The conservative values for F_V may be increased when the amount of splitting in a member is known and when no increase in splitting is expected in service. Generally the amount of splitting will be known when an existing member can be inspected.
The increase in shear capacity for reduced splitting is provided by multiplying the tabulated shear stress by the shear stress factor C_H.

9. Beam stability factor C_L

The Beam Stability factor accounts for the possibility of a beam buckling (rotating laterally) due to the compressive action in the concave portion(s) of the beam. In a great number of timber beam applications the compression zone is typically held in place against this lateral-torsion buckling by deliberate attachment of the system that the beam is supporting (floor, roof, etc.). In some cases, however, particularly with exposed, continuous beams where the compression zone is on the bottom face and is not supported laterally, potential buckling must be considered. The ‘standard’ way of dealing with this potential buckling is to determine a “Beam Stability factor” which is used as an Adjustment factor on the Bending Design Value. The Beam Stability factor reduces (reserves) the potential flexural capacity of the beam to prevent such buckling.

3.2. **Structural Glued Laminated Timber**

3.2.1. INTRODUCTION

Sawn lumber is manufactured in a large number of sizes and grades and is used for a wide variety of structural members. However, the cross sectional dimensions and lengths of these members are limited by the size of the trees available to produce this type of lumber.

When the span becomes long or when the loads become large, the use of sawn lumber may become impractical. In these circumstances structural glued laminated timber can be used.

Glulam members are fabricated from relatively thin laminations of wood. These laminations can be end-joined and glued together in such a way to produce wood members of practically any size and length. Lengths of Glulams are limited by handling systems and length restrictions imposed by highway transportation systems rather than by the size of the tree.

<u>Sizes of Glulam members</u>

The specifications for Glulam permit the fabrication of a member of any width and depth. However, standard practice has resulted in commonly accepted widths and thicknesses of laminations. The generally accepted dimensions for Glulams fabricated from the Western Species are slightly different from those for Southern Pine Glulams as given in NDS table 5.1.3. See Figure 20.

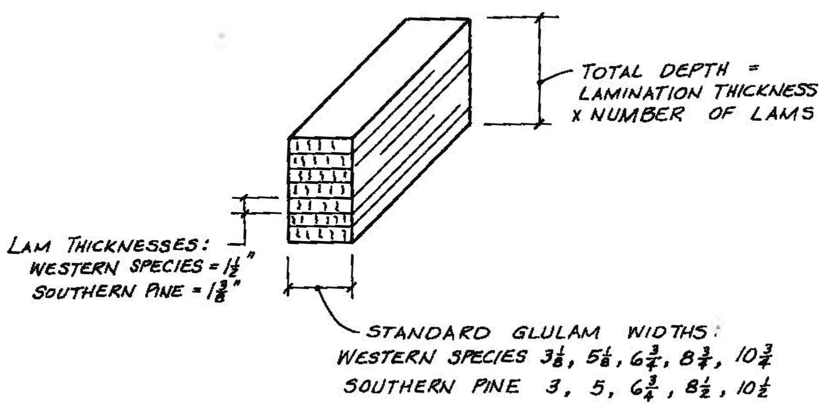

Figure 20. Difference in Glulam sizes*. Source: Breyer, Donald E. 1999.*

Because of surface requirements, Southern Pine laminations are usually thinner and narrower, although they can be manufactured to the same net sizes as Western Species if necessary. The dimensions given in Figure 20 are net sizes, and the total depth of a member will be a multiple of the lamination thickness.

3.2.2. FABRICATION OF GLULAMS

Specifications and guidelines covering the design of wood members are published by the American Institute of Timber Construction (AITC) and Engineered Wood Systems (EWS). These two associations are technical trade associations representing the structural glued laminated timber industry.
Most Gulam members are provided using Douglas Fir or Southern Pine Quality control standard ensures the production of a reliable product. In fact, the structural properties of Glulam members in most cases exceed the structural properties of sawn lumber.
The reason why the structural properties for Glulam are so high is that the material included in the member can be selected from relatively high-quality laminating stock. The growth characteristics that limit the structural capacity of a large solid sawn wood member can simply be excluded in the fabrication of a Glulam member.
In addition, laminating optimizes material use by dispersing the strength reducing effects in the laminating material throughout the member. For example, consider the laminations that are produced from a sawn lumber member with a knot that completely penetrates the member at one section. If this member were to be used to produce laminated stock which is later reassembled into a Glulam member, it is unlikely at the knot defect will be reassembled in all the laminations at exactly the same location in the Glulam member.
Besides dispersing strength-reducing characteristics, the fabrication of Glulam members makes efficient use of available structural materials in another way. High quality laminations are located in the portions of the cross section that are more highly stressed. For example, in a typical Glulam beam, wood of superior quality is located in the outer tension and compression zones, which coincides with the location of maximum bending stresses. Although the maximum bending compressive and tensile stresses are equal, research has demonstrated that the outer laminations in the tension zone are the most critical laminations in a beam. For this reason, additional grading requirements are used for the outer tension laminations.

Laminations are selected and dried to a moisture content of 16 per cent or less before gluing. Differences in moisture content for the lamination in a member should not exceed 5 per cent in order to minimize internal stresses. Because of the relatively low moisture content of Glulam members at the time of fabrication, the change in moisture content in service is generally much smaller for Glulam than it is for sawn lumber, thus Glulams are more dimensionally stable.

Two types of glue are permitted in the fabrication of Glulam members.

1. Dry-use adhesives (casein glue).

2. Wet-use adhesives (usually phenolresorcinol-base).

Both types of glue are capable of producing joints which have horizontal shear capacities in excess of the capacity of the wood itself. Although both types of glue are accepted by industry standards, currently the wet-use adhesives are used almost exclusively. This became common practice when the room-temperature-setting glues were developed for exterior use. Wet-use adhesives, as the name implies, can withstand severe conditions of exposure.

The laminations run parallel to the length of the Glulam member. The efficient use of materials and the long length of many Glulam members require that effective end splices be developed in a given lamination. While several different configurations of lamination end joint splices are possible including finger and scarf joints, virtually all glue laminated timber produced in North America uses some form of finger joint.

Although one should be aware of the basic fabrication procedures and concepts outlined in this section, the building designer does not have to be concerned about the structural design at a lamination level, in fact the manufacturer is supposed to follow a series of quality control requirements imposed by a higher institution and verified through a qualified agency to verify the adequacy of the final product.

3.2.3. GRADES OF GLULAM MEMBERS

For strength, grades of Glulam members are given as combinations of laminations. The two main types of bending combinations are bending and axial combination. In addition to grading for strength, Glulam members are graded for appearance. One of the three appearance grades (Industrial, Architectural, and Premium) should be specified along with the strength requirements to ensure that the member furnished is appropriate for the intended use. It is important to understand that the selection of an appearance grade does not affect the strength of a Glulam.

Members that are stressed principally in bending and with its load being applied perpendicular to the wide faces of laminations are produced from the bending lamination combinations. Bending combinations are defined by a combination symbol and the species of the laminating stock. The combination symbol is made of two parts. The first is the allowable bending stress for the grade in hundreds of psi followed by the letter F. For example, 24F indicates a bending combination with a tabulated bending stress of 2,400 psi for normal duration of loading and dry service conditions.
It should be noted that a number of combinations of laminations can be used to produce a given bending stress level. Therefore, there is an abbreviation that follows the bending stress level that gives the distribution of laminating stock to be used in the fabrication of a member. Two basic abbreviations are used in defining the combinations: one is for visually graded laminated stock (for example 24F-V3), and the other is for laminating stock that is mechanically graded for stiffness (for example 22F-E5) Machine stress rated (MSR) lumber is one example of E-rated laminating stock.
See Table 10 for a better understanding of Glulam bending combinations.

	Bending About X-X Axis Loaded Perpendicular to Wide Faces of Laminations					
	Extreme Fiber in Bending		Compression Perpendicular to Grain	Shear Parallel to Grain (Horizontal)	Modulus of Elasticity	Modulus of Elasticity for Beam and Column Stability
	Tension Zone Stressed in Tension (Positive Bending)	Compression Zone Stressed in Tension (Negative Bending)				
Stress Class	F_{bx}^{+} (psi)	F_{bx}^{-} (1) (psi)	$F_{c\perp x}$ (psi)	F_{vx} (4) (psi)	E_x (10^6 psi)	$E_{x\,min}$ (10^6 psi)
16F-1.3E	1600	925	315	195	1.3	0.67
20F-1.5E	2000	1100	425	210 (6)	1.5	0.78
24F-1.7E	2400	1450	500	210 (6)	1.7	0.88
24F-1.8E	2400	1450 (2)	650	265 (3)	1.8	0.93
26F-1.9E (7)	2600	1950	650	265 (3)	1.9	0.98
28F-2.1E SP (7)	2800	2300	740	300	2.1 (9)	1.09 (9)
30F-2.1E SP (7)(8)	3000	2400	740	300	2.1 (9)	1.09 (9)

Table 10. Example of Glulam bending combinations*. Source: NDS 2005 Supplement.*

Members which are principally axial load carrying members are identified with a number combination symbol such as 1, 2, 3 and so on. Because axial load members are assumed to be uniformly stressed throughout the cross section, the distribution of lamination grades is uniform throughout it, compared with the distribution of lamination quality used for beams.

Glulam combinations are, in one respect, similar to the use categories of sawn lumber. The bending combinations anticipate that the member will be used as a beam, and the axial combinations assume that the member will be loaded axially.

Bending members are fabricated with higher quality laminating stock at the outer fibers, and consequently they make efficient beams. This fact, however, does not mean that a bending combination cannot be loaded axially. Likewise, an axial combination can be designed for a bending moment. The combinations, then, have to do with the efficiency, but they do not limit the use of a member.
Design values for Glulams can be found in tables in the NDS 2005 supplement. See Table 10 above showing structural properties for members stressed primarily in bending.

3.2.4. ADJUSTMENT FACTORS

The notation for tabulated stresses, adjustment factors and tabulated stresses is essentially the same for Glulam and for sawn lumber.
The tabulated stresses for Glulams are generally larger than similar properties for sawn lumber. This is essentially a result of the selective placement of laminations and the dispersion of imperfections. However, Glulams are a wood product, and they are subject to many stress adjustment factors for sawn lumber mentioned before, and several others that are unique to Glulam design.

See Table 11 to see the different adjustment factors that apply to Glulam design:

		ASD Only	ASD and LRFD								LRFD Only		
		Load Duration Factor	Wet Service Factor	Temperature Factor	Beam Stability Factor	Volume Factor	Flat Use Factor	Curvature Factor	Column Stability Factor	Bearing Area Factor	Format Conversion Factor	Resistance Factor	Time Effect Factor
$F_{bx}' = F_{bx}^{+}$	×	C_D	C_M	C_t	C_L	C_V	C_{fu}	C_c	-	-	K_F	φ_b	λ
$F_t' = F_t$	×	C_D	C_M	C_t	-	-	-	-	-	-	K_F	φ_t	λ
$F_v' = F_v$	×	C_D	C_M	C_t	-	-	-	-	-	-	K_F	φ_v	λ
$F_{c\perp}' = F_{c\perp}$	×	-	C_M	C_t	-	-	-	-	-	C_b	K_F	φ_c	λ
$F_c' = F_c$	×	C_D	C_M	C_t	-	-	-	-	C_P	-	K_F	φ_c	λ
$F_{rt}' = F_{rt}$	×	C_D	C_M	C_t	-	-	-	-	-	-	K_F	φ_v	λ
$E' = E$	×	-	C_M	C_t	-	-	-	-	-	-	-	-	-
$E_{min}' = E_{min}$	×	-	C_M	C_t	-	-	-	-	-	-	K_F	φ_s	-

Table 11. Adjustment factors in Glulam. *Source: NDS 2005.*

A brief description of the differences and similarities of the adjustment factors for sawn lumber and Glulam is going to be given as follows. The reader should refer to section 3.1.9 for further information.

1. Wet Service Factor C_M

Tabulated design values for Glulams are for dry conditions of service. For Glulam, dry is defined as MC < 16%. For moisture contents of 16% or greater, tabulated stresses are multiplied by C_M . Values for C_M are given in tables 5A and 5B of the NDS 2005 supplement.

2. Load Duration Factor C_D

Tabulated design values for Glulam are for normal duration of load. Normal duration is defined as 10 years and is associated with floor live loads. Loads and load combinations are taken into account by multiplying by C_D. The same load duration factors are applied for sawn lumber and Glulams.

3. Flat use factor C_{fu}

The flat use factor is somewhat different for sawn lumber and for Glulam. For sawn lumber, tabulated values for F_b apply to bending about the x axis. When bending occurs about the y axis, tabulated values are multiplied by C_{fu} to convert the value to a property for the y axis.

On the other hand, Glulam members have tabulated bending values for both x and y axis. When the depth of the member for bending about the y axis is less than 12 inches, the tabulated value forF_{by} may be increased by multiplying by C_{fu}.
Because most beams are stresses about their strong axis and not the y axis, the flat use factor is not a commonly applied factor.

4. Temperature factor C_t

Tabulated design values for Glulam are for use at normal temperatures. Section 3.1.9 discussed the effects of exposure to different temperatures.

5. Volume factor C_V

It has been noted that the allowable stress in a wood member is affected by the relative size of the member. This general behavior is termed size effect. In sawn lumber, the size effect is taken into account by the size factor C_F.
In the past, the same size factor was applied to Fb for Glulam that is currently applied to the Fb for sawn lumber in the Timber sizes. Full-scale test data indicates that the size effect on a Glulam is related to the volume of the member rather to only its depth.
Therefore, the volume factor C_V replaces the size factor C_V for use in Glulam design; C_V applies only to bending stresses. Tabulated values of Fb apply to a standard-size Glulam beam with the following base dimensions:

a) Width = 5 1/8 inches.

b) Depth = 12 inches.

c) Length = 21 inches.

The volume effect factor C_V is used to obtain the allowable bending stress for other size Glulams.

4 CASE STUDY

4.1. Introduction

The purpose of this case study is to apply and expose all the mentioned concepts in this thesis. The building in question is going to be a one story regular wood framed building. The scope of this example is going to be to expose the loading acting on the building. After a load analysis has been performed on the structure, an individual member analysis using all of the concepts analyzed in the wood section will be performed. After the gravity member analysis has been performed, a seismic analysis based on the Earthquake Engineering section will be performed.

The wood rafters (or wood joists) will be the first element to be analyzed, followed by the main gravity load carrying members, which are the Glulam beams. The wood columns will be analyzed and sized after the two previous sets of members have been sized. It is important to note that the order in which these members are analyzed follows the load path from the point where the gravity loads are applied until the load transfer to the foundation.

After these three sets of members have been analyzed, we are going to go ahead and show the forces on the wood shearwalls, these are going to be the Main Lateral Force Resisting System, in other words, they are the element responsible of resisting the lateral forces and of creating an adequate load path to transfer these forces into the foundation. The shearwalls, which are going to be framed with studs spaced at approximately half a meter on center, are not going to be sized since that would be just sizing another member for bending and axial force, just like it was done with the wood rafters and the columns. The main purpose of showing the lateral load applied on them is to show how a seismic analysis is performed using the IBC 2009.

It is important to point out that from a structural point of view, the analysis that is going to be performed on this building is an extremely simplified version of what designing an entire building really is. The roof diaphragm along with its connections to the wood shear walls should be checked to provide an adequate load transfer of the lateral force. Another very important element with which the structure would fail independently of how accurate the structural design done in this thesis is the foundation. If there is not an adequate load transfer into the soil, this last one will fail in shear and the entire structure would fail. Again, this is not part of the scope of this thesis but it is important to be aware of the importance of all of these factors.

To finish with, a list of conclusions with respect to all of the applied concepts will be exposed.

<u>Design Criteria</u>

This example is going to show how different wood members are designed, and the code that is going to be followed is the NDS 2005 or National Design Specification for Wood

Construction 2005. This is the code from which the explanations of all the different factors that affect wood were gotten.

The load combination criteria are based on the IBC 2009 or International Building Code 2009. There are two different criteria when it comes to load combinations; there are load combinations at an allowable load level and a strength level. Discussing the differences between these two criteria's is not part of the scope of this thesis. Using one or the other doesn't make a difference but the allowable method will be used for consistency since the NDS has traditionally utilized allowable loads even though ultimate loads are used too and are widely accepted.

These are the load combinations that are going to be of interest in this thesis:

1. D + L
2. D + L + 0.7E

Where D is the Dead Load, L is the Live Load and E corresponds to the Earthquake load acting on the structure.

When designing any building, wind forces would be taken into consideration too. Both seismic forces and wind forces would be calculated and they would be compared. The larger force would be the one governing the lateral design. In this case study wind forces are not going to be analyzed since it is not part of the scope of this thesis but it is important to know that they should be considered.

Once all of the gravity members have been sized, a seismic analysis in accordance with the IBC 2009 will be performed. The method of determining the equivalent lateral loads was exposed in section 2.3 of this thesis. The pseudo acceleration spectrum will be calculated and plotted too.

4.2. **Building Layout**

The Building in question is going to be an L-shaped building. Its long sides are going to be 16 and 14 meters, and the open area on its corner is going to be of 6 by 8 meters.

The proposed layout is going to consist of 50x150 mm (2x6 inches) wood rafters spaced at 250mm (10 inches) on center. These rafters are going to be spanning in between Glulam beams, which are going to be 140x760mm (5.5x30 inches) with different lengths as per Figure 21. Finally, the timber posts are going to be 200x200mm (8x8 inches) and they are going to have a height of 5 meters (16.5 feet). See Figure 21 on the next page for further information on the building's layout.

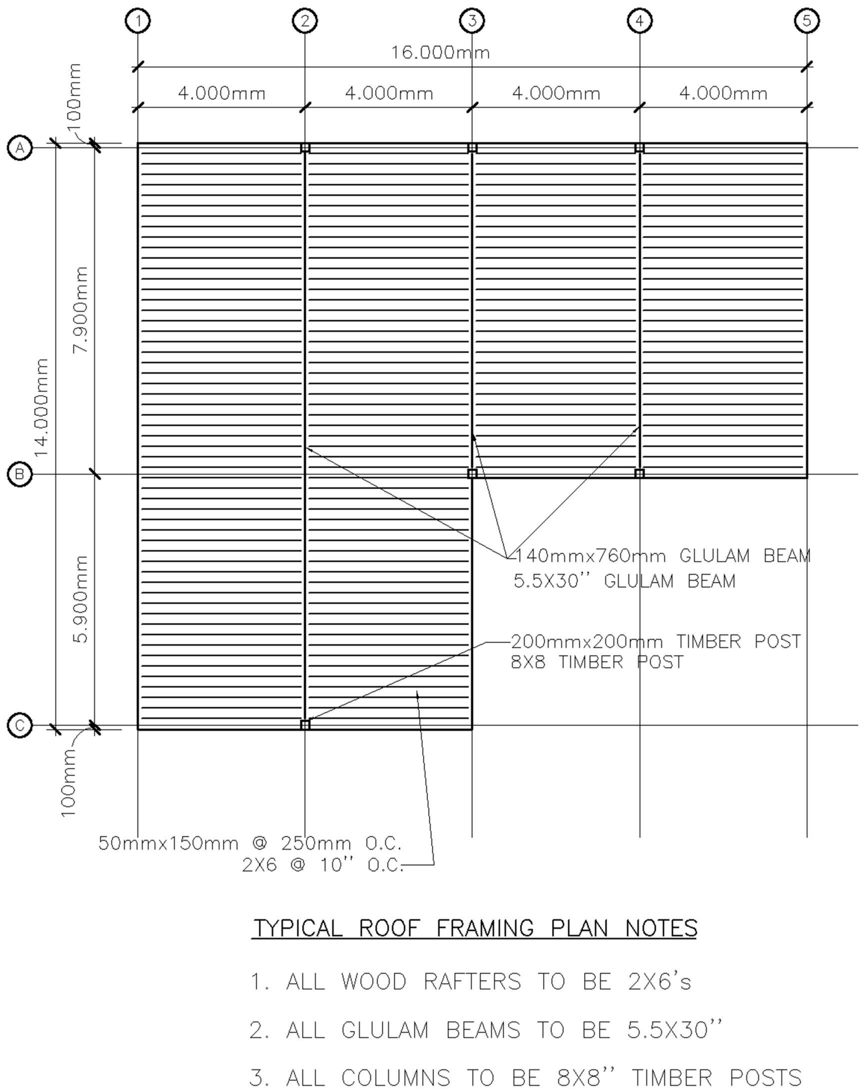

Figure 21. Building's layout and framing members. *Source: Own preparation.*

4.3. **Gravity member design**

This section is going to focus on the design of the three main gravity members which are the wood rafters, the Glulam beams and the wood columns. These three members will be sized based on the structural analysis that will be performed on them.

When designing these three members, we are going to come across different adjustment values. Most of them are going to apply to all of them, but other adjustment factors are going to apply only to the member in question. Only the most basic adjustment factors were explained in section 3. All of the additional adjustment factors not mentioned in section 3 will be commented on before the beam/column analysis is performed.

1. Load duration factor (C_D)

The load duration is going to be considered to be 10 years. Using table 2.3.2. of the NDS 2005 we have a value of $C_D = 1$.

2. Temperature factor (C_t)

The temperature at which wood is assumed to be exposed in service is going to be 25 degrees Celsius, using table 2.3.3 of the NDS 2005 this results in a value of $C_t = 1$.

3. Size factor (C_F)

The size factor is affected by the thickness, width and stress grade of the piece of lumber we are analyzing. The value for bending and compression are C_{fu} =1.15 and C_{Fc} = 1.0 can be found in tables 4B and 4D of the Annex of the NDS 2005 respectively.

4. Incising factor (C_i)

The incising factor for bending and compression can be found in table 4.3.8 of the NDS 2005 and its value is $C_i = 1$.

5. Repetitive member factor (C_r)

The repetitive member factor for the wood rafters and the wood columns is going to be $C_r = 1.15$ and $C_r = 1.0$ respectively based in table 4.3.9 of the NDS 2005.

6. Beam stability factor (C_L)

The beam stability factor is going to be assumed to be C_L = 1 for all cases because the roof diaphragm and the purlins are proving lateral support to the purlins and Glulams respectively.

We are going to assume that both sawn lumber and Glulam have a moisture content of less than 16%, that implies that the wet service factor C_M = 1 for design and won't be addressed.

All of the members designed for bending are going to be limited to a deflection of L/240, which is the maximum recommended by the IBC 2009 for roof members. This additional check is going to imply the calculation of the section's modulus of elasticity along with other sectional properties.

4.3.1. Wood rafters

Attention to a different number of factors should be paid. All of the rafters in this building are going to be spanning 4 meters (13,12ft), and that is going to be the span used to design them. We should look at the loading on the rafters: the rafters are spaced 0,25 meters on center (0,82ft) and the load that they are supporting consists of a dead load of 0,75 kN/m2(15psf) and a live load of 1kN/m2(20psf).

If we take the tributary width, which is of 0,25m into consideration, the uniformly distributed loading pattern is as follows:

Dead Load = $(0{,}75 kN/m^2)(0{,}25m)$ = $0{,}19\ kN/m$.

Live Load = $(1{,}00 kN/m^2)(0{,}25m)$ = $0{,}25\ kN/m$.

Before proceeding with the analysis, all of the reduction factors mentioned early in this thesis will be applied to the design of the wood rafters as shown in the analysis.

The load combination that is going to be used in this analysis is going to be the following:

1) D + L

Which corresponds to the full dead and live load applied simultaneously on the whole roof. This is going to be the governing load combination since for the scope of this thesis there are no lateral loads involved in the roof design.

When designing a building, the roof members are going to be part of a diaphragm, which is responsible of transferring the lateral loads to the MLFRS (Main Lateral Force Resisting System). When designing the roof diaphragm some of the roof members are going to be acting as chords and ties of the diaphragm, and consequently the load combinations that include lateral forces are going to have to be taken into consideration.

In this case, each individual rafter is going to be designed as a simply supported beam spanning in between the Glulams. Another possibility would have been to design them as continuous beams. The reason why they are going to be designed as simple span beams is because of the ease of transportation and erection. A simple span beam can be connected to the Glulams very easily with a metal hanger connector. In the US the most typical type of shear connection of this sort would be a hanger provided by Simpson, which is the most widely used manufacturer in the country.

Regarding the species of wood, Southern Pine is the species that is going to be used. The highest stress grade available is going to be used, which is Select Structural, and its structural strengths for all the different checks are going to be taken from table 8 of this thesis.

If we apply these loads to a 4-meter beam we have the following results.

<u>Structural wood Beam ANALYSIS & Design (NDS 2005)</u>

In accordance with the ASD method

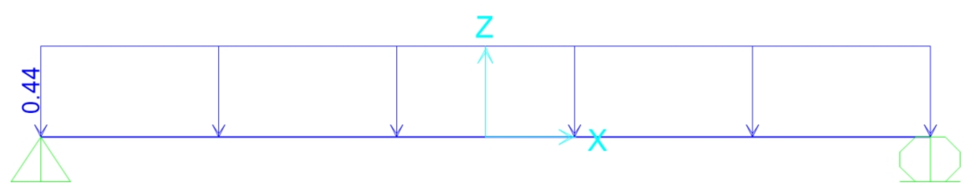

Figure 22. Rafter’s loading diagram in kN/m.*Source: SAP 2000 Beam analysis.*

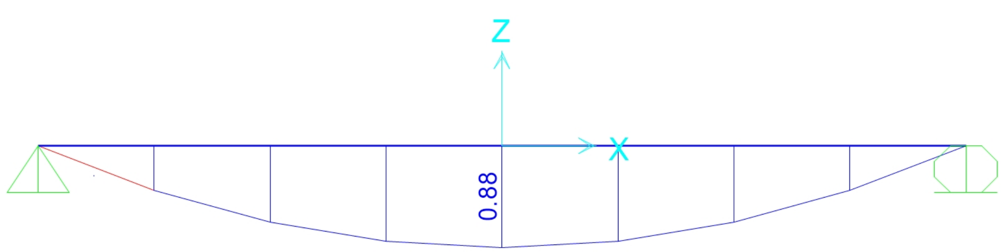

Figure 23. Rafter’s bending diagram in kNm. *Source: SAP 2000 Beam analysis.*

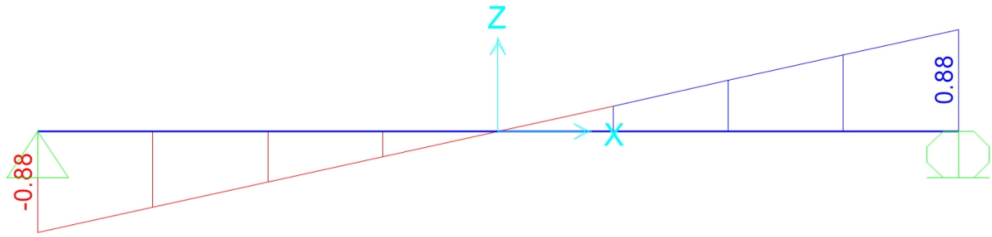

Figure 24. Rafter's shear diagram in KN.*Source: SAP 2000 Beam analysis.*

Beam loads

Dead full UDL 15 $\mathrm{lb/ft^2}$
Live full UDL 20 $\mathrm{lb/ft^2}$
Dead self weight of beam 1
Trib. width 0.82ft

Load combinations

Load combination 1	Support A	Dead	1.00
		Live	1.00
	Span 1	Dead	1.00
		Live	1.00
	Support B	Dead	1.00
		Live	1.00

Analysis results

Maximum moment;	M_{max} = 653 lb-ft (0.89kNm)	M_{min} = 0 lb-ft
Design moment;	M = max(abs(M_{max}),abs(M_{min})) = 653lb-ft	
Maximum shear;	F_{max} = 200 lb (0.89kN)	F_{min} = -200 lb
Design shear;	F = max(abs(F_{max}),abs(F_{min})) = 243 lb	
Total load on member;	W_{tot} =400 lb (1.78kN)	
Reaction at support A;	R_{A_max} = 200 lb;	R_{A_min} = 200 lb
Unfactored dead load reaction at support A;		R_{A_Dead} = 92 lb
(0.41kN)		
Unfactored live load reaction at support A;	R_{A_Live} = 107 lb (0.48kN)	
Reaction at support B;	R_{B_max} =200 lb;	R_{B_min} = 200 lb
Unfactored dead load reaction at support B;		R_{B_Dead} = 92 lb
Unfactored live load reaction at support B;	R_{B_Live} = 107 lb	

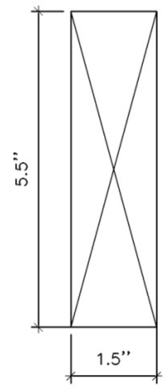

Figure 25. Rafter's cross section*. Source: Own preparation.*

Sawn lumber section details

Nominal breadth of sections;	b_{nom} = **2** in
Dressed breadth of sections;	b = **1.5** in
Nominal depth of sections;	d_{nom} = **6** in
Dressed depth of sections;	d = **5.5** in
Number of sections in member;	N = **1**
Overall breadth of member;	$b_b = N \times b$ = **1.5** in

Table 4B - Reference design values for visually graded Southern Pine dimension lumber (2"-4" thick)

Species, grade and size classification;	Southern Pine, Select Structural grade, 5"-6" wide
Bending parallel to grain;	F_b = **2550** lb/in^2
Tension parallel to grain;	F_t = **1400** lb/in^2
Compression parallel to grain;	F_c = **2000** lb/in^2
Compression perpendicular to grain;	F_{c_perp} = **565** lb/in^2
Shear parallel to grain;	F_v = **175** lb/in^2
Modulus of elasticity;	E = **1800000** lb/in^2
Mean shear modulus;	$G_{def} = E / 16$ = **112500** lb/in^2

Member details

Service condition;	**Dry**
Load duration;	**Ten years**

The beam is one of three or more repetitive members

Section properties

Cross sectional area of member;	$A = N \times b \times d$ = **8.25** in^2
Section modulus;	$S_x = N \times b \times d^2 / 6$ = **7.56** in^3
	$S_y = d \times (N \times b)^2 / 6$ = **2.06** in^3
Second moment of area;	$I_x = N \times b \times d^3 / 12$ = **20.80** in^4
	$I_y = d \times (N \times b)^3 / 12$ = **1.55** in^4

Adjustment factors

Load duration factor - Table 2.3.2;	C_D = **1.00**
Temperature factor - Table 2.3.3;	C_t = **1.00**
Size factor for bending - Table 4B;	C_{Fb} = **1.00**
Size factor for tension - Table 4B;	C_{Ft} = **1.00**

Size factor for compression - Table 4B; C_{Fc} = **1.00**
Flat use factor - Table 4B; C_{fu} = **1.15**
Incising factor for modulus of elasticity - Table 4.3.8; C_{iE} = **1.00**
Incising factor for bending, shear, tension & compression - Table 4.3.8
C_i = **1.00**
Repetitive member factor - cl.4.3.9; C_r = **1.15**
Bearing area factor - cl.3.10.4; C_b = **1.00**
Depth-to-breadth ratio; d_{nom} / (N b_{nom}) = **3.00**
- Beam is fully restrained
Beam stability factor - cl.3.3.3; C_L = **1.00**

Strength in bending - cl.3.3.1
Design bending stress; $F_b' = F_b\ C_D\ C_t\ C_L\ C_{Fb}\ C_i\ C_r$ = **3372** lb/in^2
Actual bending stress; $f_b = M / S_x$ = **1036**lb/in^2
f_b / F_b' = **0.31**

PASS - Design bending stress exceeds actual bending stress

Deflection - cl.3.5.1
Modulus of elasticity for deflection; $E' = E\ C_{ME}\ C_t\ C_{iE}$ = **1800000** lb/in^2
Design deflection; δ_{adm} = **0.0043**in L_{s1} = **0.677** in
Bending deflection; δ_{b_s1} = **0.660** in
Shear deflection; δ_{v_s1} = **0.012** in
Total deflection; $\delta_a = \delta_{b\_s1} + \delta_{v\_s1}$ = **0.672** in
δ_a / δ_{adm} = **0.992**

PASS - Design deflection is less than total deflection

4.3.2. GLULAM BEAMS

The Glulam beam with the longest span is going to be 14 meters (45,93ft). The wood type that is going to be used for its design is going to be Southern Pine. In regards to the loading, the reactions from the wood rafters are going to be the applied loads on the Glulam beam. The rafters are closely spaced enough so that its load can be put in as a uniformly distributed load. We have the following loading for the Glulam beam:

Dead Load = $(0{,}75 kN/m^2)(4m)$ = $3\ kN/m$.

Live Load = $(1{,}00 kN/m^2)(4m)$ = $4\ kN/m$.

Just like it was done for the wood rafters before, the Glulam beams are going to be designed as simply supported beams spanning in between columns. The adjustment factors that are going to be used for the beam design in bending are going to be the load duration factor (C_D), the temperature factor (C_t), the flat use factor (C_{fu}), the curvature factor (C_C), and the volume factor (C_V).

The volume factor (C_V) is calculated using the formulas shown in appendix 5 of the NDS 2005 and it is shown in the beam analysis.

The flat use factor (C_{fu})is always > 1 for Glulam beams. In Glulam design its common practice to omit this value since it is conservative and that is what is going to be done in this example.

The curvature factor (C_C) only applies to beams with curvature so its value it is going to be 1 in this example.

The glulam design is most likely going to be governed by deflection since the span is very large.

Applying these loads to a Glulam beam we have the following analysis:

Structural Glued laminated timber (Glulam) Beam ANALYSIS & Design (NDS 2005)

In accordance with the ASD method

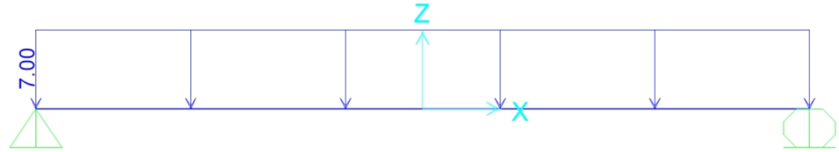

Figure 26. Glulam's loading diagram in kN/m. *Source: SAP 2000 Beam analysis.*

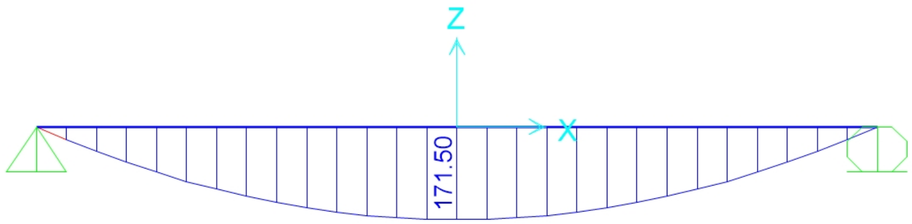

Figure 27. Glulam's bending diagram in kNm. *Source: SAP 2000 Beam analysis.*

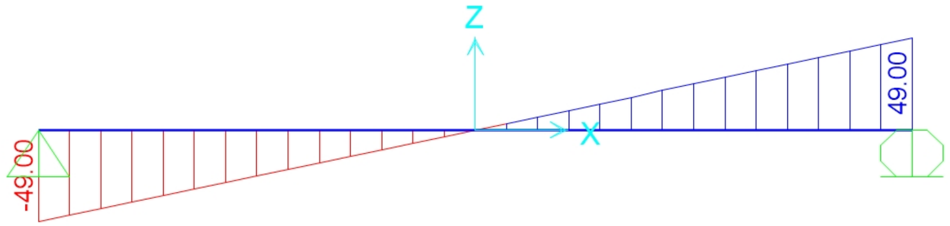

Figure 28. Glulam's shear diagram in kN. *Source: SAP 2000 Beam analysis.*

Applied loading

Beam loads

Dead full UDL 205 lb/ft (3kN/m)
Live full UDL 274 lb/ft (4kN/m)
Dead self weight of beam 1

Load combinations

Load combination 1	Support A	Dead	1.00
		Live	1.00
	Span 1	Dead	1.00
		Live	1.00
	Support B	Dead	1.00
		Live	1.00

Analysis results

Maximum moment; M_{max} = **126491 lb-ft (171.50 kNm)** M_{min} = **0** lb_ft

Design moment; M = max(abs(M_{max}),abs(M_{min})) = **126491 lb-ft**

Maximum shear; F_{max} = **11016 lb (49.00 kN)** F_{min} =**11016** lb

Design shear; F = max(abs(F_{max}),abs(F_{min})) = **11016 lb**

Reaction at support A; R_{A_max} = **11016 lb**; R_{A_min}= **11016 lb**

Unfactored dead load reaction at support A; R_{A_Dead} = **4721 lb (21.00kN)**

Unfactored live load reaction at support A; R_{A_Live} = **6295 lb(28.00kN)**

Reaction at support B; R_{B_max} = **11016 lb;** R_{B_min}=**-11016 lb**

Unfactored dead load reaction at support B; R_{B_Dead} =**4721 lb**

Unfactored live load reaction at support B; R_{B_Live} = **6295 lb**

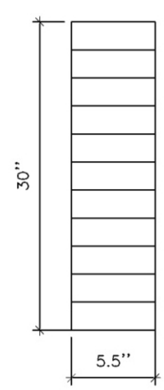

Figure 29. Glulam's cross section*. Source: Own preparation.*

Glulam section details

Net finished breadth of sections; $b = \mathbf{5.5}$ in
Net finished depth of sections; $d = \mathbf{30}$ in
Number of sections in member; $N = \mathbf{1}$
Overall breadth of member; $b_b = N \times b = \mathbf{5.5}$ in
Alignment of laminations; **Horizontal**

Table 5A - Reference design values for structural glued laminated softwood timber combinations

Stress class; 24F-1.8E
Bending parallel to grain; $F_b = \mathbf{2400}$ lb/in^2
Tension parallel to grain; $F_t = \mathbf{1100}$ lb/in^2
Compression parallel to grain; $F_c = \mathbf{1600}$ lb/in^2
Compression perpendicular to grain; $F_{c\_perp} = \mathbf{650}$ lb/in^2
Shear parallel to grain; $F_v = \mathbf{265}$ lb/in^2
Modulus of elasticity; $E = \mathbf{1800000}$ lb/in^2
Mean shear modulus; $G_{def} = E / 16 = \mathbf{112500}$ lb/in^2

Member details

Service condition; **Dry**
Length of bearing; $L_b = \mathbf{4}$ in
Load duration; **Ten years**
The beam is one of three or more repetitive members

Section properties

Cross sectional area of member; $A = N \times b \times d = \mathbf{165.00}$ in^2
Section modulus; $S_x = N \times b \times d^2 / 6 = \mathbf{825.00}$ in^3
$S_y = d \times (N \times b)^2 / 6 = \mathbf{151.25}$ in^3
Second moment of area; $I_x = N \times b \times d^3 / 12 = \mathbf{12375.00}$ in^4
$I_y = d \times (N \times b)^3 / 12 = \mathbf{415.94}$ in^4

Adjustment factors

Load duration factor - Table 2.3.2; $C_D = \mathbf{1.00}$
Temperature factor - Table 2.3.3; $C_t = \mathbf{1.00}$
Length of beam between points of zero moment; $L_0 = \mathbf{45.932}$ ft
For Southern Pine; $x = \mathbf{20}$
Volume factor - eq.5.3-1; $C_V = \min((21\text{ ft} / L_0)^{1/x} \times (12\text{ in} / d)^{1/x} \times (5.125\text{ in} / b)^{1/x}, 1) = \mathbf{0.92}$
Depth-to-breadth ratio; $d / (N \times b) = \mathbf{5.45}$
- Beam is fully restrained

Beam stability factor - cl.3.3.3; $C_L = \mathbf{1.00}$

Strength in bending - cl.3.3.1

Design bending stress; $F_b' = F_b\ C_D\ C_t\ C_V\ C_L = \mathbf{2197}$ lb/in^2
Actual bending stress; $f_b = M / S_x = \mathbf{153.32}$ lb/in^2
$f_b / F_b' = \mathbf{0.07}$

PASS - Design bending stress exceeds actual bending stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection; $E' = E_x\ C_{ME}\ C_t = \mathbf{1800000}$ lb/in^2
Design deflection; $\delta_{adm} = \mathbf{0.0043}$in $L_{s1} = \mathbf{2.370}$ in
Bending deflection; $\delta_{b\_s1} = \mathbf{2.244}$ in
Shear deflection; $\delta_{v\_s1} = \mathbf{0.102}$ in
Total deflection; $\delta_a = \delta_{b\_s1} + \delta_{v\_s1} = \mathbf{2.346}$ in
$\delta_a / \delta_{adm} = \mathbf{0.990}$

PASS - Design deflection is less than total deflection

4.3.3. WOOD COLUMNS

The wood columns are going to have a height of 5 meters (16.4ft), and the loading that they are going to carry corresponds to the Glulam reactions. Just like it was done for the previous cases, Southern Pine is the wood type that is going to be used for the column design along with an unbraced length of 5-meters.

No lateral forces are going to be applied to the column since it is considered not to have any lateral rigidity. That means that the lateral loads are going to only go into the elements that have lateral rigidity, which in this case are the wood shearwalls.

All of the adjustment factors mentioned at the beginning of this section are going to be used in addition to the column stability factor (C_p). Its value can be calculated using equation 3.7-1 of the NDS 2005 as shown in the column analysis below.

The column carries the following axial loading proportional to its tributary area:

Dead Load = (0,75kN/m2)(4m)(14/2m) = 21 kN(4721 lbs).

Live Load = (1kN/m2)(4m)(14/2m) = 28 kN(6295lbs).

Structural wood member designStructural wood member designStructural wood Beam Design (NDS 2005)

In accordance with the ASD method

Analysis results

Design axial compression; P = **11016** lb

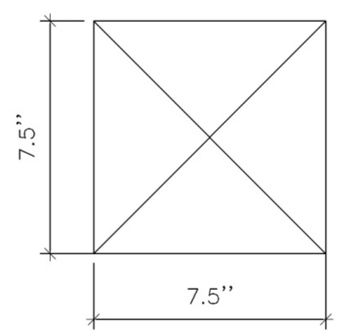

Figure 30. Column's cross section.*Source: Own preparation.*

Sawn lumber section details

Nominal breadth of sections; b_{nom} = **8** in
Dressed breadth of sections; b = **7.5** in
Nominal depth of sections; d_{nom} = **8** in
Dressed depth of sections; d = **7.5** in
Number of sections in member; N = **1**
Overall breadth of member; $b_b = N \times b$ = **7.5** in

Table 4D - Reference design values for visually graded timbers (5"x 5" and larger)

Species, grade and size classification; Southern Pine, Select Structural grade, Posts and timbers
Bending parallel to grain; F_b = **1500** lb/in^2
Tension parallel to grain; F_t = **1000** lb/in^2
Compression parallel to grain; F_c = **950** lb/in^2
Compression perpendicular to grain; F_{c_perp} = **375** lb/in^2
Shear parallel to grain; F_v = **165** lb/in^2
Modulus of elasticity; E = **1500000** lb/in^2
Mean shear modulus; G_{def} = E / 16 = **93750** lb/in^2

Member details

Service condition; **Dry**
Load duration; **Ten years**
Unbraced length in x-axis; L_x = **16.4** ft
Effective length factor in x-axis; K_x = **1**
Effective length in x-axis; $L_{ex} = L_x \times K_x$ = **16.4** ft
Unbraced length in y-axis; L_y = **16.4** ft
Effective length factor in y-axis; K_y = **1**
Effective length in y-axis; $L_{ey} = L_y \times K_y$ = **16.4** ft

Section properties

Cross sectional area of member; $A = N \times b \times d$ = **56.25** in^2
Section modulus; $S_x = N \times b \times d^2 / 6$ = **70.31** in^3
$S_y = d \times (N \times b)^2 / 6$ = **70.31** in^3
Second moment of area; $I_x = N \times b \times d^3 / 12$ = **263.67** in^4
$I_y = d \times (N \times b)^3 / 12$ = **263.67** in^4

Adjustment factors

Load duration factor - Table 2.3.2; C_D = **1.00**
Temperature factor - Table 2.3.3; C_t = **1.00**
Size factor for bending - Table 4D; C_{Fb} = **1.00**
Size factor for tension - Table 4D; C_{Ft} = **1.00**
Size factor for compression - Table 4D; C_{Fc} = **1.00**
Flat use factor - Table 4D; C_{fu} = **1.00**
Incising factor for bending, shear, tension & compression - Table 4.3.8
C_i = **1.00**
Repetitive member factor - cl.4.3.9; C_r = **1.00**
Adjusted modulus of elasticity for column stability; $E_{min}' = E_{min}\ C_{ME}\ C_t\ C_i$ = **470000** lb/in^2
Reference compression design value; $F_c = F_c\ C_D\ C_{Mc}\ C_t\ C_{Fc}\ C_i$ = **950** lb/in^2
Critical buckling design value for compression; $F_{cE} = 0.822\ E_{min}' / (L_{ex} / d)^2$ = **561** lb/in^2
c = **0.80**
Column stability factor - eq.3.7-1
$C_P = (1 + (F_{cE} / F_c)) / (2\ c) - [((1 + (F_{cE} / F_c)) / (2\ c))^2 - (F_{cE} / F_c) / c]$ = **0.54**

Depth-to-breadth ratio; d_{nom} / (N x b_{nom}) = **1.00**
- Beam is fully restrained

Beam stability factor - cl.3.3.3; C_L = **1.00**

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress; $F_c' = F_c\ C_D\ C_t\ C_{Fc}\ C_i\ C_P$ = **509** lb/in^2

Applied compressive stress; $f_c = P / A$ = **195** lb/in^2

f_c / F_c' = **0.38**

PASS - Design compressive stress exceeds applied compressive stress

The analysis performed can be summarized in the table shown below. DCR refers to the demand to capacity ratio of the structural member in question.

Structural Member	Design stress(psi)	Allowable Stress (psi)	DCR	Size(in)
Rafters	1036	3372	0.31	2x6
Glulams	153.32	2197	0.07	5.5x30
Posts	195	509	0.38	8x8

Table 12. Summary of gravity analysis on structural members*. Source: Own preparation from analysis results.*

4.4. **Seismic Analysis and Shear Walls**

In this section a seismic analysis based on the IBC 2009 is going to be performed. The purpose is to come up with a lateral load applied to the roof and then transferred into the shearwalls, which are the elements responsible of transferring this lateral load right into the foundation.

For practical purposes, we are going to assume that the structure is located in a high risk seismic area, and both the short and 1 second response coefficients are going to be 3.0g and 1.5g respectively. These same values are 1.5g and 0.6g in San Francisco, California, which is considered a high-risk seismic area so that would put our structure in an even worse position.

The pseudo acceleration spectrum is going to be shown and the buildings natural period will be calculated and inserted in the graph, which will give us a design pseudo acceleration value based on the natural period of vibration of the structure in question.

From the analysis of the nature of seismic forces we know that one of the most critical factors in determining the seismic force in a structure is the seismic weight, which corresponds to the weight tributary to the structure that contributes to the applied forces at the roof. It can also be defined as the force that stresses the structure and that is transferred into the foundation. That means that the different elements that are going to contribute are the entire roof weight and only half of the weight of all of the walls since the bottom part is transferred to the foundation without stressing the structure.

The seismic weight is broken down as follows:

1. Roof Dead Load

Roof = $(0{,}75kN/m^2)[(14)(16) - (6)(8)] = 132kN.$

Where$0.75KN/m^2$ corresponds to the pressure applied by the dead load on the roof and the second term corresponds to the area of the roof. Multiplying these two terms we obtain the total contribution of the roof to the total dead load of the structure.

2. Walls Dead Load

Walls = $(0{,}75kN/m^2)[(2)(5m)(16m) + (2)(5m)(14m) + (5m)(6m) + (5m)(8m)]/2 = 139kN.$

Where $0.75KN/m^2$ corresponds to the wall's pressure dead load and the second term corresponds to the area of the walls. By multiplying these two terms we obtain the total dead load on the walls. As mentioned before, we divide by 2 to just account for the part that actually stresses the structure.

This gives us the total seismic weight of:

$132kN + 139kN = 271\,kN\,(61\,kips).$

All of the information necessary to come up with an equivalent lateral force on the structure in question has been provided. The remainder of this section will be divided in 2 different parts:

1. Generation of the design response spectrum.

2. Calculation of the equivalent lateral force using the method explained in section 2.3.2.

The design response spectrum will be plotted and its features will be discussed. The first step is calculating the parameters that define the spectrum. All of the equations exposed in section 2.3 will be used.

The first step to generate the design spectrum is to calculate the design pseudo accelerations. To obtain the design pseudo accelerations values for the short and 1-second period structure, the modification factors for the type of soil have to be taken into account. The IBC 2009 recommends using Site Class D, which corresponds to a stiff soil profile, when no geotechnical information on the soil is provided. That is what was done in this particular case.

The response spectrum is going to be plotted until a period of 2 seconds, since our structure will clearly be very far away from this limit. Only high structures with large natural periods of vibration fall on that side of the spectrum.

The following is a summary of all the factors (which are code referenced) used to obtain the design pseudo acceleration values:

Seismic Parameters (ASCE 7-05 Design)

Site parameters

Site class: D

Mapped acceleration parameters (Section 11.4.1)

at short period; $S_S = 3$

at 1 sec period; $S_1 = 1.5$

Site coefficient; Table 11.4-1;Table 11.4-2;

at short period (Table 11.4-1); $F_a = 1.0$

at 1 sec period (Table 11.4-2); $F_V = 1.5$

Spectral response acceleration parameters

at short period (Eq. 11.4-1); $S_{MS} = F_a S_S = 3.000$

at 1 sec period (Eq. 11.4-2); $S_{M1} = F_V S_1 = 2.250$

Design spectral acceleration parameters (Sect 11.4.4)

at short period (Eq. 11.4-3); $S_{DS} = 2 / 3\ S_{MS} = 2.000$

at 1 sec period (Eq. 11.4-4); S_{D1} = 2 / 3 S_{M1} = 1.500

The design pseudo acceleration values have been calculated. The only information left to generate the design spectrum is to calculate the key natural periods in the spectrum.

Applying the same equations previously mentioned in section 2.3 we obtain the design response spectrum:

$$T_0 = 0.2\frac{S_{D1}}{S_{DS}} = 0.2\frac{1.5}{2.0} = 0.15s$$

$$T_S = \frac{S_{D1}}{S_{DS}} = \frac{1.5}{2.0} = 0.75s$$

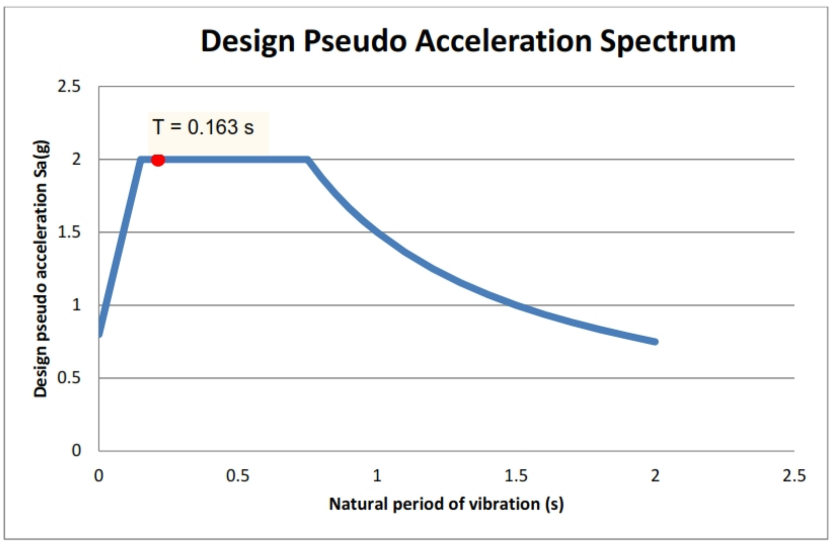

Figure 31. Design response spectrum.*Source: Own preparation from analysis results.*

Now that the design spectrum has been calculated and plotted we will proceed and use the “Equivalent Force Resisting Method” so we can come up with an equivalent lateral force applied to our structure. Like it was mentioned before, section 2.3.2 will be used to compute this force.

We can see that if we enter the graph with a natural period of vibration of T=0,163s, which corresponds to the period of the structure in question and will be calculated in the next page, we get the maximum value of the design pseudo acceleration which is $S_{DS} = 2.0s$ since the peak value is reached at a period of 0,15 seconds.

The following analysis will determine our lateral equivalent lateral force:

Seismic Forces (ASCE 7-05)

Design spectral acceleration parameters (Sect 11.4.4)

at short period (Eq. 11.4-3); S_{DS} = 2.0g

at 1 sec period (Eq. 11.4-4); S_{D1} = 1.50g

Seismic risk category; II

Approximate fundamental period

Height above base to highest level of building; h_n = 16.4ft (5m)

From Table 12.8-2:

Structure type; All other systems

Building period parameter; C_t = 0.02

Building period parameter; x = 0.75

Approximate fundamental period (Eq 12.8-7); $T_a = C_t\ (h_n)$x1sec / (1ft)x= 0.163 sec

Building fundamental period (Sect 12.8.2); T = Ta = 0.163 sec

Seismic response coefficient

Seismic force-resisting system (Table 12.14-1); B BUILDING FRAME SYSTEMS

23. Light-framed walls sheathed with wood structural panels rated for shear resistance

Response modification factor (Table 12.14-1); R = 7

Seismic importance factor (Table 11.5-2); Ie = 1.000

Seismic response coefficient (Sect 12.8.1.1)

Calculated (Eq 12.8-2); $C_{S\text{-}CALC} = S_{DS} / (R / Ie)$=0.286

Maximum (Eq 12.8-3); $C_{S\text{-}MAX} = S_{D1} / (T\ (R / Ie))$ = 1.315

Minimum:

Eq 12.8-5 $C_{S\text{-}MIN1}$ = max(0.044 SDS Ie,0.01) = 0.088

Eq 12.8-6 $C_{S\text{-}MIN2}$= (0.5S1) / (R / Ie) = 0.107

$C_{S\text{-}MIN}$ = 0.107

Seismic response coefficient; C_S= 0.286

Seismic base shear (Sect 12.8.1)

Effective seismic weight of the structure;   W = 61.0 kips

Seismic response coefficient; C_S = 0.286

Seismic base shear (Eq 12.8-1);   V = C_SW = 17.4 kips

All of the equations exposed above are numbered in reference to the ASCE 7-05.

It should be noted that the Occupancy Category, which determines the value of the importance factor is going to be assumed to be II (see Table 6). This is going to result in a value of I of 1.

Also, the response modification factor, which represents the level of ductility given to our structure, is going to be R=7 per table 12.4-1 of the ASCE 7-05, see Table 13. This table is very long and only the part of interest (Item 23) will be shown.

Taking all of these factors into consideration and following all the steps of the “Equivalent Lateral Force Method” we come up with the following:

The total seismic force applied to the structure in question is going to be of 17,4 kips or 77,4kN. This force is going to be resisted by the shearwalls proportionally to the tributary width affecting each one of them. That is, each shear wall would be design as a cantilever beam with a point load on its tip, and that point load would correspond to the lateral load tributary to it.

Seismic Force–Resisting System	ASCE 7 Section where Detailing Requirements are Specified	Response Modification Coefficient, R[a]	System Overstrength Factor, Ω_0[g]	Deflection Amplification Factor, C_d[b]	Structural System Limitations and Building Height (ft) Limit[c]				
					Seismic Design Category				
					B	C	D[d]	E[d]	F[e]
22. Prestressed masonry shear walls	14.4	$1\frac{1}{2}$	$2\frac{1}{2}$	$1\frac{3}{4}$	NL	NP	NP	NP	NP
23. Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1, 14.1.4.2, and 14.5	7	$2\frac{1}{2}$	$4\frac{1}{2}$	NL	NL	65	65	65
24. Light-framed walls with shear panels of all other materials	14.1, 14.1.4.2, and 14.5	$2\frac{1}{2}$	$2\frac{1}{2}$	$2\frac{1}{2}$	NL	NL	35	NP	NP
25. Buckling-restrained braced frames, non-moment-resisting beam-column connections	14.1	7	2	$5\frac{1}{2}$	NL	NL	160	160	100
26. Buckling-restrained braced frames, moment-resisting beam-column connections	14.1	8	$2\frac{1}{2}$	5	NL	NL	160	160	100
27. Special steel plate shear wall	14.1	7	2	6	NL	NL	160	160	100
C. MOMENT-RESISTING FRAME SYSTEMS									
1. Special steel moment frames	14.1 and 12.2.5.5	8	3	$5\frac{1}{2}$	NL	NL	NL	NL	NL
2. Special steel truss moment frames	14.1	7	3	$5\frac{1}{2}$	NL	NL	160	100	NP
3. Intermediate steel moment frames	12.2.5.6, 12.2.5.7, 12.2.5.8, 12.2.5.9, and 14.1	4.5	3	4	NL	NL	35[h,i]	NP[h]	NP[i]
4. Ordinary steel moment frames	12.2.5.6, 12.2.5.7, 12.2.5.8, and 14.1	3.5	3	3	NL	NL	NP[h]	NP[h]	NP[i]

Table 13. Design coefficients for Seismic Force Resisting Systems. *Source: ASCE 7-05.*

The most important seismic parameters and the result of the seismic analysis can be summarized in the table shown below. The first to terms correspond to the mapped and design pseudo acceleration values for short periods. R refers to the response modification factor and C_S corresponds to the seismic response coefficient. The last two terms correspond to the seismic weight and the seismic base shear respectively. The most important seismic parameters involved in the seismic analysis are summarized in Table 14.

$S_S(g)$	$S_{DS}(g)$	R	C_S	$W(kN)$	$V(kN)$
3	2	7	0.286	271	77,4

Table 14. Seismic analysis summary.*Source: Own preparation from analysis results.*

4.5. **Conclusions**

Summarizing, what was done here was a gravity and seismic analysis of a single story wood framed building with the following conclusions:

Wood can be used as a structural material for every single element of a building (besides the foundation), going from the wood rafters, through the main gravity supporting members, into the foundation. The Main Lateral Resisting members can also be made of wood just like the plywood and studs in a shear wall like it was done in this case study. These last two weren't analyzed in this thesis due to their complexity.

From a structural point of view, using wood instead of concrete or steel can be a little more tedious due to the large number of adjustment factors that have to be computed for each member, whereas coming up with a structural capacity for steel or concrete is normally way quicker. Another feature that lengthens the process too is having different unfactored strength values depending on the orientation and nature of the property in question.

Contrary to belief, wood can handle big loads and large spans thanks to wood engineered products. The Glulam beam sized in the case study was designed to span 14 meters, which is not very common for steel or concrete in single story residential or retail buildings. Engineered wood can be used for long spans such as large steel and concrete members would when working with these types of buildings. The same thing happens with the columns, the worst loaded column resists a force of 49kN which is not considered extremely high for a column but there is a wide selection of bigger wood column sizes that would allow us to resist higher loads.

Performing a seismic analysis on a simple, regular shaped building is not extremely tedious and complicated. The codes make most of the work already providing simple formulas that take into consideration all of the parameters involved in the seismic analysis, therefore, generating the design pseudo acceleration spectrum is not really necessary. The reason why it was generated in this case study is just to show how the code enters the design spectrum based on the structures natural period of vibration.
The problem of coming up with an equivalent seismic force comes down to mapping the short and 1 second acceleration values and following a series of steps that will lead us to an equivalent seismic force.

5 CONCLUSIONS

5.1. Main conclusions

The last section of this thesis is going to address the general conclusions after completing this thesis;

One of the most important conclusions of this thesis is the versatility and usefulness of wood as a structural engineering material. Granted that wood doesn't perform as well as steel or concrete in high-rise buildings, it offers a very competitive performance when talking about one or two story residential and retail buildings. The biggest drawback for wood in these types of building is its durability problem, which can be avoided by providing the right chemical treatments and performing an adequate maintenance.

From an economical point of view, even if wood is a very competitive material for the type of construction mentioned in the previous paragraph, it won't make sense to use it if there is not a close enough source that allows the price of wood to be competitive with respect to steel and concrete. For example, the US and Canada have a very large amount of forests, that means that they can both offer a very large supply of wood, which make it a very attractive engineering material to be used in these areas. In the US, forests are especially abundant in the Pacific Northwest, making the material cheaper in the states located in that area with respect to other states which are further apart from a wood source.

From a structural engineering point of view, wood is a very different material than steel or concrete. Steel and concrete's strength capacities are well defined through several formulas given a series of parameters, but that is not the case with wood.

Wood's strength capacities depend on a large number of factors; the material strength depends on the orientation of the fibers, with its strength being reduced when load is applied perpendicular to them. Another factor would be the wood species, as well as the stress grade and all of the modification factors. This makes wood calculations a little confusing to the engineer who is not used to designing wood members.

Wood is a light, potentially inexpensive material that can be very suitable in certain applications and when used wisely, can be even more economic and perform better than regular steel and concrete in construction. Special consideration should be taken into all the hardware involved in all of the wood connections. The hardware wasn't analyzed in this thesis but it is important when performing a real wood building analysis.

From an environmental point of view, there is no better engineering material than wood. Wood is a renewable material and it contributes negatively to the greenhouse effect since it stores carbon in it. On the other hand, steel and concrete are a lot harsher on the environment since they contribute positively to the greenhouse effect as they release a lot of carbon to the atmosphere through their fabrication processes and the maintenance implied in their use.

Earthquake Engineering is a crucial design consideration in certain parts of the world. Seismic forces can be considerably larger than wind forces, that is, if not considered in design, the consequences would be catastrophic in the event of an earthquake. It is very important to understand that seismic loads depend on the type of structure we are designing and on its fundamental period. A certain structure, for instance, a gas canopy, can be located in an extremely high risk seismic zone and not have its lateral designed governed by seismic forces. In this case, since the structure would be very light and have a big wind area of exposure due to its fascia, it is likely that wind will govern the design.

The fundamentals of seismic design have been explained and all of the steps to come up with a final equivalent seismic force have been shown too. Earthquake Engineering is complex science and has a lot of factors involved, spanning from the nature of the zone where the structure would be located, to the geological formations and the nature of the structure to be designed. Small changes in the factors concerning seismic design can change the lateral forces considerably. For instance, a structure can behave very differently to an earthquake depending on its geometry. Adding or subtracting a single floor to a low-rise building can change dramatically the way this last one behaves under the same earthquake. That can be easily seen when looking at the design pseudo acceleration spectrum, where its value can vary a lot depending on the range of the spectrum we are in. Therefore, understanding the main factors that are involved in the computation of seismic forces is extremely important, and that was one of the main purposes of this thesis. Tables from the IBC 2009 were provided to show where all of the different factors contributing to the lateral design were taken from.

All in all, taking into consideration the limitations and the scope of work, this thesis has shown me all the steps to size a generic structural member with wood. Setting aside the complexity in the structural analysis, after completing this thesis I would feel comfortable sizing any bending or axial member for a given set of design loads. Also, after explaining and applying the step by step seismic analysis, a seismic analysis on any building with similar features to the one analyzed in this thesis can be performed and broken down to computing a series of factors and using a series of formulas, making the seismic analysis simpler to the design engineer.

5.2. **Future directions in wood design**

The wood industry is not a static business. If there is a better way of doing something, a better way of doing something will be found. The term “better” typically refers to more accurate methods of structural design, but there is an underlying economic force that drives the system.

Many wood based products that were unavailable only a few years ago are in widespread use today. These include a number of structural use panels, wood I joists, laminated veneer lumber and resawn Glulam beams among others. Most of these developments are a result of new technology, and they represent an economic response to environmental concerns and resource constraints. With the use of these products, the move is plainly in the direction of engineered wood construction. All of these wood engineered products offer a solution to the limitations that sawn lumber has such as its size and strength. The use of these wood engineered products has let us reach longer spans and has provided us with the versatility that other structural materials have developed, putting wood in the front line in innovation and obviously in sustainable design.

The design profession is caught in the middle of this development spiral too. The NDS has had broad changes in its design criteria in the last years. Recent changes include new lumber values from the In-Grade testing programs, new column and laterally unbraced beam formulas, new interaction formulas for members with combined stresses and an engineering mechanics approach to the design of wood connections, which haven't been analyzed in this thesis.

The current NDS is based on a deterministic method known as allowable stress design (ASD), where computed stresses are based on working or service loads. Another approach to design is based on reliability theory, which is commonly referred to as limit states design or resistance factor a load design (LRFD). This last approach is generally agreed upon by the profession as the appropriate technique for use in structural design. Reinforced concrete has used this last method for quite some time, while the structural steel industry currently recognizes both ASD and LRFD formats. However, as experience is gained, it is expected that the LRFD will eventually become the accepted design method for structural steel.

The wood industry is also in the process of moving to an LRFD format. The NDS just recently included the LRFD method as an alternate way to design with wood. Though this change, ASD will continue to be the popular method in the near future.

In regards to an extension of this thesis, a thorough analysis on wood connections could be made. The focus of this thesis was introducing the reader to wood and its properties from the most basic level, giving guidance on how to size a member for axial and flexural stresses. The following step would be analyzing more complex wooden elements such as the different elements and behavior of a wood diaphragm or shearwall, spanning from its members to how they are connected in order to transfer the loads to the Main Lateral Force Resisting System or MLFRS. As in all structural systems, connections are a crucial design consideration and therefore should be analyzed when designing with wood. The design of wood connections consists of designing the hardware to be able to transfer the design loads. This hardware may consists of screws, bolts, lag screws and a wide variety of metal connectors. A more detailed analysis on these is encouraged by the writer as a way to enhance our knowledge on wood.

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